APPENDIX F
GEOLOGIC HAZARDS EVALUATIONS

F-1. Introduction

This appendix describes guideline procedures for the evaluation of seismic-geologic site hazards, other than the ground shaking hazard. These hazards include: (a) surface fault rupture; (b) soil liquefaction; (c) soil differential compaction; (d) landsliding; and (e) flooding. The evaluations of the hazards described in this appendix should be carried out by qualified geotechnical professionals. Depending on the hazard, disciplinary expertise in geotechnical engineering, geology, and seismology may be needed.

a. Overview of process for conducting geologic hazards evaluations. The process described herein for seismic-geologic hazards evaluation is a two-step process—screening and evaluation. If a significant hazard is disclosed by this process, then hazard remediation should be developed.

b. Organization of remainder of this appendix. Paragraph F-2 describes and illustrates the geologic hazards. Screening procedures for these hazards are presented in paragraph F-3. The intent in the screening process is to utilize readily available data and criteria to ascertain whether a significant potential for any of the hazards exists at the site. Paragraph F-4 presents hazard evaluation procedures in the event that the screening process results in a conclusion that more detailed evaluation is required to assess the hazard and its significance. Paragraph F-5 provides preliminary information regarding hazard mitigation. Requirements for documentation of the evaluations of geologic hazards are described in paragraph F-6. Examples of geologic hazard evaluations are presented in Appendix G.

F-2. Description of Geologic Hazards

The following paragraphs provide brief descriptions of the seismic-geologic hazards of surface fault rupture, soil liquefaction, soil differential compaction, landsliding, and flooding. Hazard significance in terms of potential ground movements and effects on structures are also summarized.

a. Surface fault rupture. Earthquakes are caused by the sudden slip or displacement along a zone of weakness in the earth's crust, termed a fault. Surface fault rupture is the manifestation of the fault displacement at the ground surface for those cases where the fault slip extends to the ground surface. Generally, fault rupture extends to the ground surface only during moderate- to large-magnitude earthquakes (magnitudes equal to or greater than 6). However, not all moderate-to large-magnitude earthquakes produce fault slip at the ground surface. In some cases, the fault displacement may occur entirely at depth, with little or no apparent permanent surface deformation (e.g., 1989 Loma Prieta, California earthquake of moment magnitude 7.0), or with more subdued or diffuse surface warping and fracturing (as may have accompanied the 1994 Northridge, California earthquake of moment magnitude 6.7).

(1) Mode of fault movement. The mode of surface fault deformation is influenced by the type of faulting. Different types of faults are illustrated in Figure F-1. These types are distinguished by the primary sense of relative displacement between the two sides of the fault. Strike-slip faults are characterized by horizontal movement; reverse or thrust faults involve relative upward movement of the crustal block above the fault plane; normal faults involve relative downward movement of the block above the fault plane; and oblique faults are characterized by both strike-slip and reverse or normal types of movement.

(2) Magnitude of displacements. Surface fault displacements may range from a fraction of an inch to several feet or more depending on the earthquake magnitude, steepness of the fault plane, type of movement, and other factors. These same factors, as well as the nature of the surface geologic materials, also influence how wide the zone of surface rupture is likely to be. Because fault displacements tend to occur abruptly, often across a narrow zone, surface fault rupture can be catastrophic to structures situated directly astride the rupture zone. Figure F-2 illustrates surface fault rupture that occurred in the 1992 Landers, California earthquake. During this moment magnitude 7.3 earthquake, the displacement was mainly of the strike-slip type (see Figure F-1) and the maximum observed horizontal displacement along the fault was 5.5 m (18 feet). Figure F-3 illustrates damage to a structure astride the surface

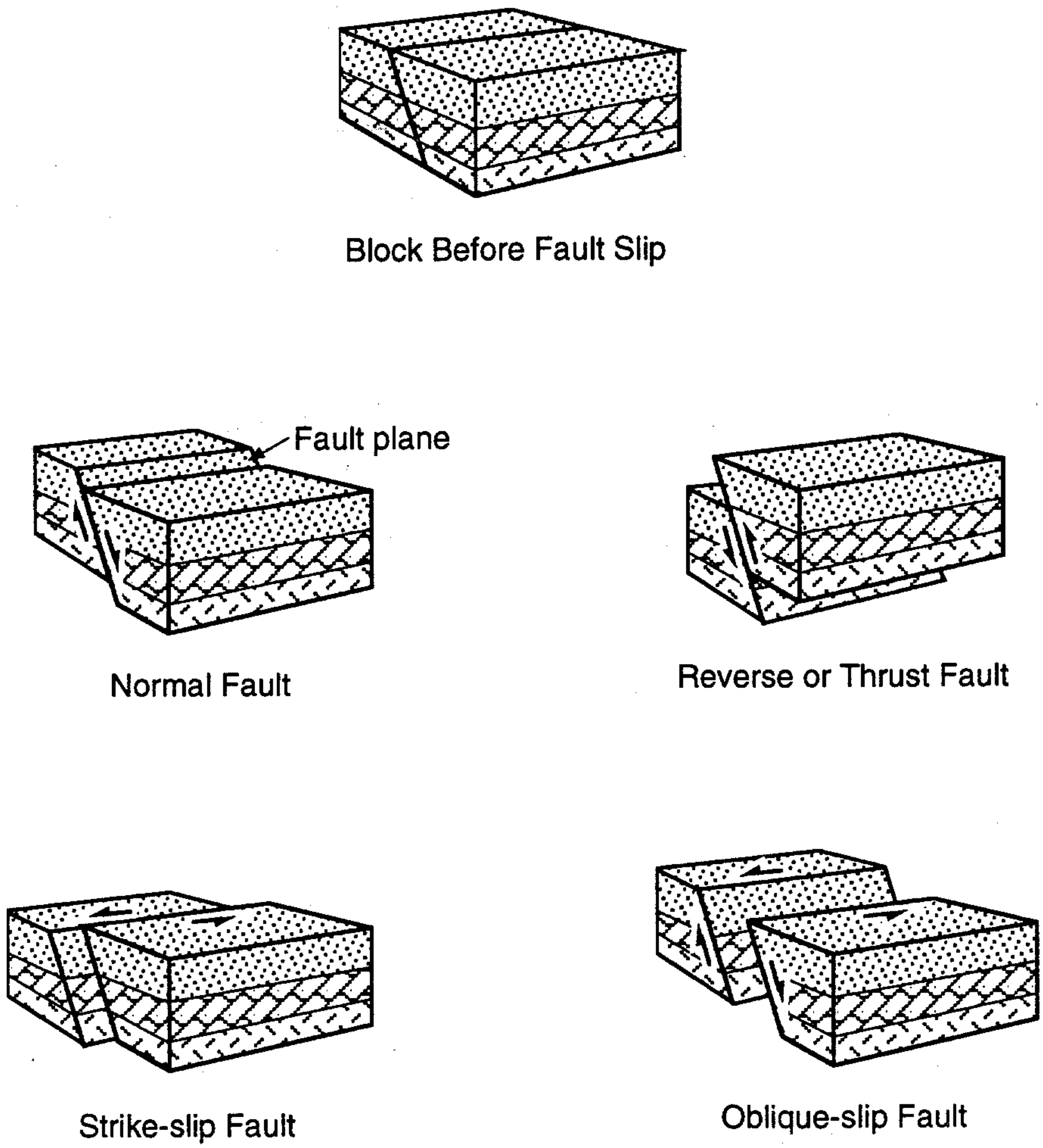


Figure F-1 Types of faults.

Figure F-2 Surface faulting accompanying Landers, California earthquake of June 28, 1992.

Figure F-3 House damaged by ground displacement caused by surface faulting accompanying the San Fernando, California earthquake of February 9, 1971.

fault rupture of the 1971 San Fernando, California, earthquake (moment magnitude 6.6), which was of the reverse- or thrust-fault type (see Figure F-1). More than 1.8 m (6 feet) of combined vertical and horizontal displacement occurred along the surface trace of the fault during the San Fernando earthquake.

b. Soil liquefaction. Soil liquefaction is a phenomenon in which a soil deposit below the groundwater table loses a substantial amount of strength due to strong earthquake ground shaking. The reason for the strength loss is that some types of soil tend to compact during earthquake shaking and this tendency for compaction will induce excess pore water pressures which, in turn, causes strength reduction in the soil. Recently deposited (i.e. geologically young) and relatively loose natural soils and uncompacted or poorly compacted fills are potentially susceptible to liquefaction. Loose sands and silty sands are particularly susceptible. Loose silts and gravels also have potential for liquefaction. Dense natural soils and well-compacted fills have low susceptibility to liquefaction. Clay soils are generally not susceptible, except for highly sensitive clays found in some geographic locales.

(1) Potential consequences of liquefaction include: (1) reduction or loss of foundation bearing strength, which can lead to large structure settlements due to shear failure in the weakened soils; (2) flotation of lightweight structures embedded in liquefied soil; (3) differential compaction, due to soil densification as excess pore water pressures dissipate, that can lead to structure differential settlement; (4) horizontal movements due to lateral spreading or flow sliding of liquefied soils, which can lead to total and differential lateral movements of structures; and (5) increased lateral pressures on retaining walls for liquefied soils. Other manifestations of liquefaction can also occur and may or may not pose a risk to structures. Sand boils are common surface manifestations of liquefaction, in which the liquefied soil under pressure is ejected to the ground surface through a vent and forms a conical-shaped "sand boil" deposit around the vent. Although sand boils are usually not a cause of damage to structures, the ejection of subsurface materials in a sand boil may pose a settlement hazard to an immediately adjacent structure. Another phenomenon accompanying liquefaction is ground oscillation, in which the ground overlying liquefied soil experiences large-displacement transient oscillations that can result in extensional and compressional ground failures such as opening and closing of fissures, buckling of sidewalks, thrusting of sidewalks and curbs over streets, breakage of utility lines, and the like.

(2) Figure F-4 illustrates the consequence of loss of foundation bearing capacity that occurred during the 1964 Niigata earthquake in Japan. As shown, apartment buildings experienced large settlements and tilts due to liquefaction of the underlying soil.

(3) Liquefaction-induced lateral movements can occur on extremely flat slopes, less than 1 percent in some cases. The potential for lateral movements is increased if there is a "free face," such as a river channel or the sloping shoreline of a lake or bay, toward which movements can occur. The hazard of lateral spreading is illustrated diagrammatically in Figure F-5. Figure F-6 illustrates the effect of lateral spreading on a building during the 1989 Loma Prieta earthquake; the movements pulled the structure apart.

c. Soil differential compaction. Differential compaction refers to the densification of soils that may occur due to strong earthquake ground shaking. As noted above, densification can occur with time following liquefaction as soil excess pore water pressures dissipate. In soils that are above the groundwater table and thus not susceptible to liquefaction, densification can occur as the strong ground shaking occurs. Loose natural soils and uncompacted and poorly compacted fills are susceptible to densification. If densification does not occur uniformly over an area, the resulting differential settlements can be damaging to structures. In general, the amounts of movement associated with the hazard of differential compaction are less than those due to liquefaction-induced bearing capacity failure or lateral spreading.

d. Landsliding. Landsliding can occur due to the loss of soil strength accompanying liquefaction, as mentioned above. However, landsliding can also occur in soils and rocks on hillside slopes in the absence of liquefaction, due to the inertia forces induced by the ground shaking. Consequences of landsliding include differential lateral and vertical movements of a structure located within the landslide zone, or landslide debris impacting a structure located below a landslide. An example of a structure within a zone of earthquake-induced

Figure F-4 Bearing capacity failure due to liquefaction, Niigata, Japan earthquake of June 16, 1964.

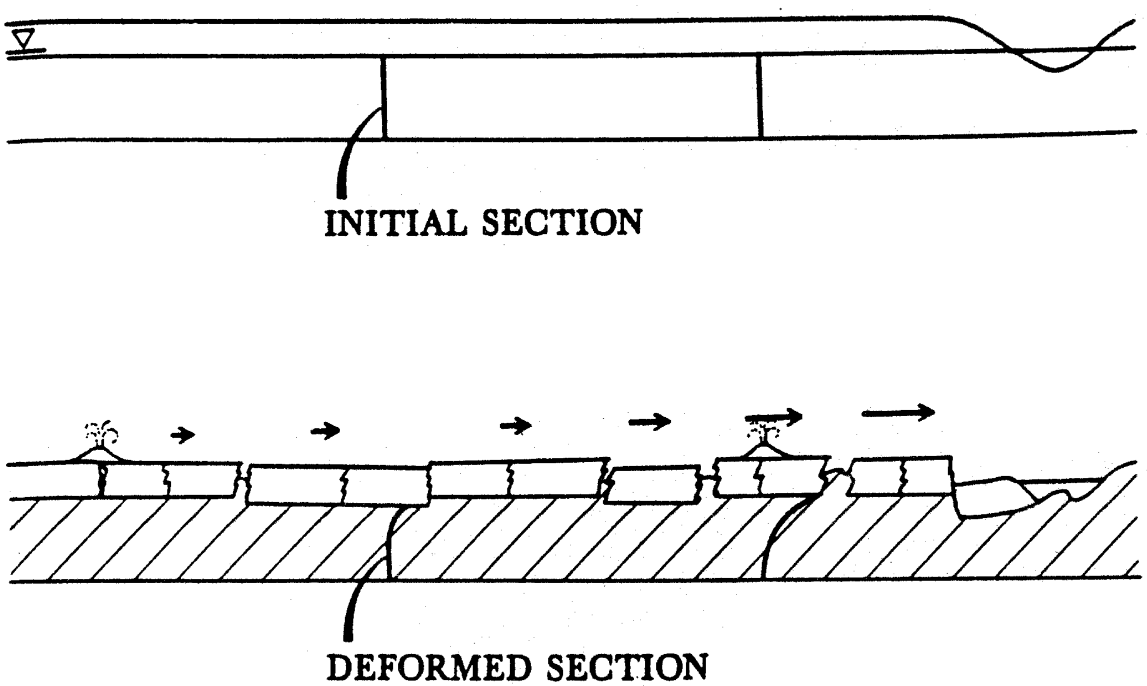


Figure F-5 Diagram of lateral spread before and after failure. Liquefaction occurs in the cross-hatched zone. The surface layer moves laterally down the mild slope, breaking up into blocks bounded by fissures. The blocks also may tilt and settle differentially with respect to one another (from Youd, 1984; National Research Council, 1985).

Figure F-6 Lateral spreading failure due to liquefaction, University of California Marine Laboratory Building at Moss Landing, Loma Prieta, California earthquake of October 17, 1989.

landsliding is shown in Figure F-7. Figure F-8 illustrates the hazard of landslide material (rockfall debris in this case) impinging on a structure below a slope. Even a single large boulder dislodged from a slope can cause considerable damage to a structure below.

e. Flooding. Earthquake-induced flooding at a site can be caused by a variety of phenomena including seiche, tsunami, landsliding, and dam, levee, and water storage tank failures. Seiches are waves induced in an enclosed body of water such as a bay, lake, or reservoir by interaction of the water body with the arriving seismic waves. Seiches can be caused by earthquakes that occur either in the region of a site or thousands of miles away. Seiche waves may reach several feet in height and can be damaging to facilities located at or very near the shoreline.

(1) Tsunamis are ocean waves generated by vertical seafloor displacements associated with large offshore earthquakes. Tsunami waves at a site may be produced by local or distant earthquakes; and wave heights may reach tens of feet at some coastal locations. Onshore tectonic movements accompanying earthquakes can also cause flooding, such as crustal tilting causing water to overflow a dam or uplift along a thrust fault causing damming of a river.

(2) Another source of tsunami waves is rapid landsliding into bodies of water, either from hillside slopes above the water body or from submarine slopes within the water body. Another type of flooding hazard is that caused by earthquake-induced failure of a dam, levee, or water storage tank.

F-3. Screening Procedures

The following sections describe screening procedures for the geologic hazards described above. The possible conclusions from screening for each hazard are: (1) a significant hazard potential does not exist; or (2) further evaluation (described in paragraph F-4) is required to assess the hazard and its significance. There are two screening procedures that should be followed for all the hazards. First, a check should be made as to whether a hazard has previously occurred at the site (or in the near vicinity of the site in similar geotechnical conditions) during historical earthquakes. This check may involve review of the earthquake history of an area, review of published post-earthquake reconnaissance reports, and discussions with engineers and geologists knowledgeable of the prior earthquake performance of an area. Although such information does not exist for all locations, it is available for numerous locations throughout the country; for example, in Northern California (Youd and Hoose, 1978; Tinsley et al., 1994); in the New Madrid, Missouri region (Obermeier, 1989; Wesnousky et al., 1989); in the Charleston, South Carolina region (Obermeier et al., 1986; Gohn et al., 1984); in the northeastern United States (Tuttle and Seeber, 1989); among others. If a hazard has previously occurred at the site, then the evaluations described in paragraph F-4 should be conducted; its absence, however, does not preclude the occurrence of the hazard during future seismic events. Second, a check should be made as to whether the site is included in an area for which a regional earthquake hazard map has been prepared by a federal or state agency. For example, under the U.S. Geological Survey's National Earthquake Hazard Reduction Program (NEHRP), liquefaction potential maps have been prepared for several urban areas of the United States. If the area containing the site has been mapped as having a high risk with respect to any geotechnical hazard (e.g., in an area of "high liquefaction potential"), then evaluations described in paragraph F-4 should be conducted.

a. Surface fault rupture. The potential for experiencing fault rupture (or not) at a site is controlled primarily by the regional and local tectonic environment. For the hazard of surface fault rupture to be present, an active fault or faults must pass beneath the site. A fault is considered to be active and capable of producing surface rupture if the fault exhibits any of the following characteristics indicative of recent tectonic activity:

$ It is a documented source of historical earthquakes or is associated spatially with a well-defined pattern of microseismicity.

$ Its trace (the zone where the fault intersects the ground surface) is marked by well-defined geomorphic features like scarps, deflected drainages, closed depressions, etc. that are suggestive of geologically recent faulting. Because such features are easily modified or destroyed by erosion and deposition, their

Figure F-7 House and street damaged by several inches of landslide displacement caused by the San Fernando, California earthquake of February 9, 1971.

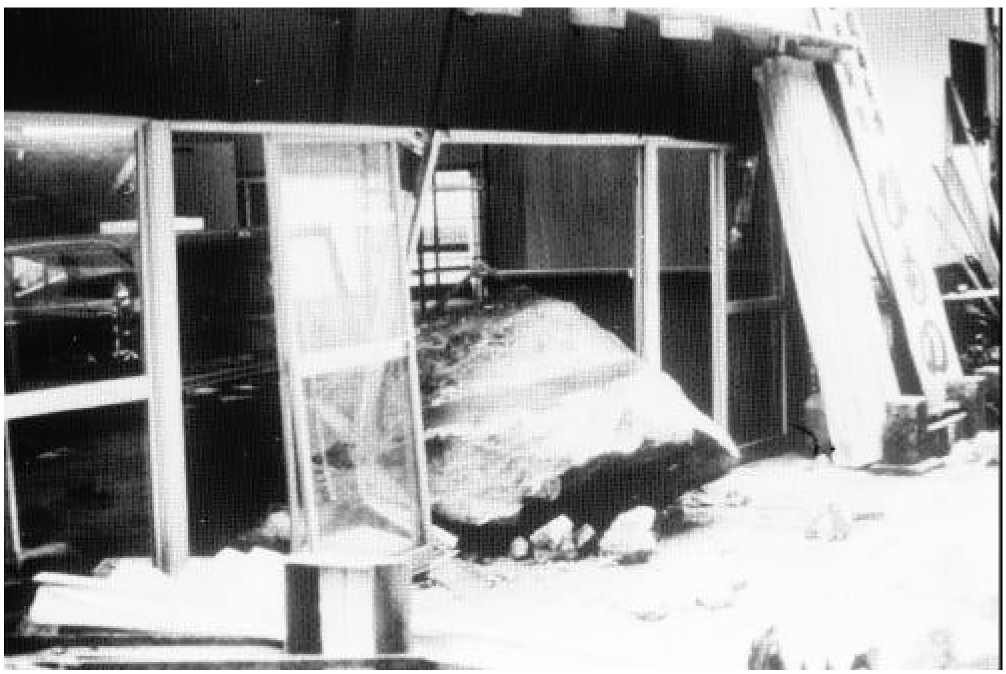


Figure F-8 Damage to store front caused by rock fall during the San Fernando, California earthquake of February 9, 1971.

presence in the landscape indicates geologically recent tectonic activity.

- $ It has experienced at least one episode of surface rupture (including fault creep) during approximately the past 11,000 years (Holocene time) or multiple episodes of rupture during the last 100,000 years (the late Quaternary period).

(1) Regional potential for surface fault rupture. The potential for surface fault rupture varies greatly in different parts of the United States. The potential exists mainly along and near the active deformation boundary between the North American and Pacific tectonic plates, which extends along coastal California, Oregon, Washington, and southeastern and southern Alaska. The tectonic effects of this plate boundary, including surface faulting, extend to the eastern margin of the Rocky Mountains. Beyond the plate boundary, intraplate earthquakes occur within the North American plate but generally have not been accompanied by surface fault rupture. In the eastern United States, the only active faults that have been mapped at the ground surface to date are the Meers and Criner faults in southern Oklahoma. These faults, which comprise two segments of the Frontal Wichita Fault System, have well developed geomorphic expression and geologically documented episodes of slip during Holocene time. Intraplate earthquakes within the Pacific plate occur beneath the state of Hawaii and are triggered by the underground movement of basaltic magma from which the island volcanoes have been built. Ground fissuring can occur due to the swelling of volcanoes prior to eruption.

(2) Steps involved in screening. Screening for surface fault rupture should include:

- $ A review of geologic maps available from the U.S. Geological Survey, state geological agencies, and local government agencies. The geologic maps typically show the location of faults and identify the ages of the geologic units displaced by the fault. Large-scale geologic maps (e.g., 1:24,000 or larger scale) prepared within the last 30 years generally provide the most reliable information for this type of assessment. In California, "Alquist-Priolo" maps, published by the California Division of Mines and Geology, define those zones within the state in which surface fault rupture is a significant risk. The U.S. Geological Survey in Denver is currently preparing maps that show the major active faults in the Western Hemisphere. In the process of obtaining and reviewing these maps, government geologists who may be actively working on the geology of the area including the site should be contacted as needed.

- $ A review of topographic maps available from the U.S. Geological Survey. These maps depict the topography in the general site vicinity and can be used to identify geomorphic features that might indicate the presence of faults.

- $ A reconnaissance of the site and review of aerial photographs. With respect to the surface fault rupture hazard, a site reconnaissance and review of available aerial photographs, aimed at detecting geologic or geomorphic evidence of faulting, should be conducted if adequate geologic and topographic maps are not available.

(3) Screening criteria. It can be assumed that a severe hazard due to surface fault rupture does not exist at the site if, based upon a review of the available information, both of the following screening criteria are met:

(a) Geologic and topographic maps show no faults passing beneath the site or in the vicinity of the site; or if the maps show faults and folds in the vicinity of the site, the geologic maps and related cross sections clearly show that earth materials that are as least as old as Quaternary (1.8 million years old), like soils, alluvium, terrace surfaces and/or deposits, lie across the folds and faults and are not deformed by them.

(b) Site reconnaissance and air photo review does not detect evidence of faulting at the site.

(4) Example. An example of screening for surface fault rupture potential is given in Appendix G.

b. Soil liquefaction. The potential for experiencing liquefaction (or not) at a site during an earthquake is primarily influenced by the characteristics of the subsurface soils (e.g., geologic age and depositional environment, soil type, density), the depth to the groundwater table, and the amplitude and duration of ground shaking. As such, these factors can provide a basis for evaluating a site for liquefaction hazard. For screening level evaluations, criteria are given for assessing subsurface soils and groundwater information available for a site. Screening criteria are not made a function of ground shaking level because current understanding of liquefaction behavior does not preclude its occurrence at any ground shaking level, although there are no reported/known cases of historical liquefaction for peak ground accelerations less than about 0.07g.

(1) Sources of information. Sources of available information to be reviewed in conducting a screening evaluation for liquefaction hazard include:

- Geologic maps - Large-scale (e.g., 1:24,000) or smaller-scale (e.g., 1:250,000) geologic maps are generally available for many areas from geologists of regional U.S. Geological Survey offices, state geological agencies, or local government agencies. The geologic maps typically identify the age, depositional environment, and material type for a particular mapped geologic unit.

- Topographic maps - Similar availability as geologic maps. These maps depict the general slope gradient and direction for the general site vicinity and the presence of any significant nearby free-face. Site grading plans may also be available for review.

- Boring logs - Foundation engineering reports prepared for a facility typically contain logs of geotechnical borings drilled at the site. The logs typically contain information regarding the stratigraphy (soil type), penetration resistance (density) and the depth at which groundwater was encountered. The foundation engineering reports may also contain laboratory test data such as grain size distributions, Atterberg limits, unit weights, shear strength, etc.; these data are commonly reported on the boring logs and reflected in the soil descriptions given on the logs. In the absence of site-specific boring logs, logs for borings drilled on an adjacent site may provide useful screening information, as may logs of water wells drilled on site or nearby. If off-site information is utilized, it is important to examine the appropriateness of the off-site data by checking the mapped geologic similarity of the sites (see above).

- Groundwater depth - The depth of the groundwater table below the existing ground surface is commonly reported on boring logs or water well logs; regional groundwater depth (elevation) contour maps may also be available and utilized if site-specific or nearby measurements are not. Possible seasonal and historic fluctuations of groundwater levels should also be reviewed/considered.

- Building foundation - Available drawings and other information on the proposed building foundation system should be reviewed to ascertain the type and depth of foundation (e.g., spread footings, piles).

- Site ground reconnaissance - Walkdown of the site and buildings should be conducted to observe and note the existing characteristics of the site (e.g., topography, especially slopes or free faces). During the site reconnaissance, observations of ground distress and/or building distress at the site and nearby sites that may be related to geotechnical processes should also be recorded.

(2) Screening criteria. It can be assumed that a significant hazard due to liquefaction does not exist at a site if, based on the review of available information, one of the following screening criteria is met:

(a) The geologic materials underlying the site are either bedrock or have a very low liquefaction susceptibility according to the relative susceptibility ratings that Youd and Perkins (1978) assigned based upon general depositional environment and geologic age of the deposit. These susceptibility ratings are shown in Table F-1.

(b) The soils below the groundwater table at the site are: stiff clays or clayey silts and have a clay content (grain size< 0.005 mm or 0.0002 inches) greater than 15 percent, liquid limit greater than 35 percent, or natural moisture content less than 90 percent of the liquid limit (Seed and Idriss, 1982); or cohesionless soils (i.e. clean or silty sands, silts, or gravels) with a minimum normalized Standard Penetration Test (SPT) resistance, $(N_1)_{60}$, value of 30 blows/0.3 m (30 blows/foot); or cohesionless

Table F-1 Estimated susceptibility of sedimentary deposits to liquefaction during strong ground motion (after Youd and Perkins, 1978).

Type of Deposit	General Distribution of Cohesionless Sediments in Deposits	Likelihood that Cohesionless Sediments, When Saturated, Would be Susceptible to Liquefaction (by Age of Deposit)			
		<500 yr Modern	Holocene >11 ka	Pleistocene 11 ka - 2 Ma	Pre-Pleistocene >2 Ma
(a) Continental Deposits					
River channel	Locally variable	Very high	High	Low	Very low
Floodplain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plain	Widespread	Moderate	Low	Low	Very low
Marine terraces and plains	Widespread	---	Low	Very low	Very low
Delta and fan-delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low
(b) Coastal Zone					
Delta	Widespread	Very high	High	Low	Very low
Estuarine	Locally variable	High	Moderate	Low	Very low
Beach					
High wave-energy	Widespread	Moderate	Low	Very low	Very low
Low wave-energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Fore shore	Locally variable	High	Moderate	Low	Very low
(c) Artificial					
Uncompacted fill	Variable	Very high	---	---	---
Compacted fill	Variable	Low	---	---	---

soils that classify as clayey sand (SC) or clayey gravel (GC) with $(N_1)_{60}$ greater than 20. (The parameter $(N_1)_{60}$ is defined in paragraph F-4.) However, cohesive soils that are highly sensitive based on measured soil properties or local experience are not screened out. To be classified as highly sensitive, a soil must possess each of the following property values: sensitivity greater than 4; liquid limit less than 40%; moisture content greater than 0.9 times the liquid limit; liquidity index greater than 0.6; and $(N_1)_{60}$ less than 5 or normalized cone penetration resistance, q_{c1}, less than 1 MPa (20 ksf). Areas of the U.S. with known highly sensitive soils include some coastal areas of Alaska, along the St. Lawrence River, some eastern and western coastal areas with estuarine soil deposits, and near saline lakes in the Great Basin and other arid areas. (Refer to Youd, 1998).

(c) The groundwater table is at least 15 m (49 feet) below the ground surface, including considerations for seasonal and historic groundwater level rises, and any slopes or free-face conditions in the site vicinity do not extend below the groundwater elevation at the site.

(3) Example. An example of screening for the hazard of liquefaction is given in Appendix G.

c. Soil differential compaction. Information sources to be reviewed in conducting a screening evaluation for differential compaction are the same as those identified above for the liquefaction potential hazard. The site reconnaissance observations for the liquefaction potential hazard can be used for the screening of the hazard of differential compaction.

(1) Screening criteria. It can be assumed that a significant hazard due to differential compaction does not exist if the soil conditions meet both of the following criteria:

(a) The geologic materials underlying foundations and below the groundwater table do not pose a significant hazard due to liquefaction.

(b) The geologic materials underlying foundations and above the groundwater table are either: Pleistocene in geologic age (older than 11,000 years); stiff clays or clayey silts; or cohesionless sands, silts, and gravels with a minimum $(N_1)_{60}$ of 20 blows/0.3 m (20 blows/foot).

d. Landsliding. The potential for landsliding or downslope movement is dependent on slope geometry, subsurface soil, rock and groundwater conditions, past slope performance, and level of ground shaking. The screening procedures involve a review of geologic and topographic maps, review of available data on the subsurface conditions, and performing reconnaissance of the site and adjacent areas. Review of available aerial photographs is desirable, especially if adequate geologic and topographic maps are not available. In some areas, governmental agencies have prepared slope stability maps showing existing landslides and/or relative slope stability. These should be reviewed if available. If appropriate, geologists and engineers in government agencies knowledgeable of the performances of slopes in the area should be contacted.

(1) Screening criteria. It can be assumed that a significant hazard due to earthquake-induced landsliding does not exist if all of the following criteria are satisfied:

(a) The building site is not located within a pre-existing active or ancient landslide, and there are no landslides on slopes of similar geometry and geology in the site vicinity. The site is not located on, above, or below a slope that displays cracking or other signs of actual or incipient slope movement. There is not an obvious hazard to the building from falling rocks or shallow soil flows on slopes located above the building.

(b) The site is not located adjacent to a shoreline.

(c) The site is not located in a zone that has been mapped as having a high landslide potential (static or seismic).

(d) The building is located above a slope, is a horizontal distance of at least three times the slope height from the toe of the slope, and is set back a distance at least equal to the slope height from the top of the slope. The geologic materials in the slope are stiff cohesive (and nonsensitive) clays or clayey silts, dense sands that do not have a significant liquefaction potential, or bedrock. There are no obvious planes of weakness in the slope, such as bedding planes dipping out of the slope. If fill is present in the slope, there is evidence that it has been engineered, well compacted, and placed with engineering inspection and testing.

(e) The building is located below a slope, is a horizontal distance of at least twice the slope height from the toe of the slope, and the slope is underlain by geologic materials as stated in (d) above.

(2) Example. An example of screening for the hazard of landsliding is given in Appendix G.

e. Flooding. The hazard of flooding due to many causes, including tsunami, seiche, tectonic movements, and failure of water retention structures can be assumed to be not significant if the facility is not located near a body of water nor in an area that could be inundated by the hazard.

(1) Tsunami and seiche. For facilities located near coastal waters, the hazard of tsunami due to earthquake-induced seafloor displacements can be assumed to be not significant if the ground surface elevation of the facility above sea level is greater than the estimated potential maximum tsunami wave height as given in Figure F-9. Although records of seiche occurrence are relatively incomplete, it would appear to be rare for a seiche wave to exceed about 2 m (7 feet) in height. Therefore, the seiche hazard can be screened out for sites located more than 2 m (7 feet) above the adjacent water body.

(2) Landsliding-induced tsunami. The potential for rapid hillside landsliding into bodies of water can be assumed to be not significant if slopes in similar geologic materials in the vicinity have performed well historically and the slopes are not oversteepened. If similar slopes and geologic formations extend underwater, they are also unlikely to be susceptible to significant submarine landsliding. Loose or soft submarine deposits such as deltaic deposits could be susceptible to rapid landsliding.

(3) Flooding due to tectonic movements. The potential for flooding due to tectonic movements can be assumed to be not significant if the regional faults would not be expected to produce tectonic movements to a degree that could interact with water bodies and cause flooding. Such judgements should be made by experienced geologists or seismologists who are knowledgeable of the regional tectonic setting.

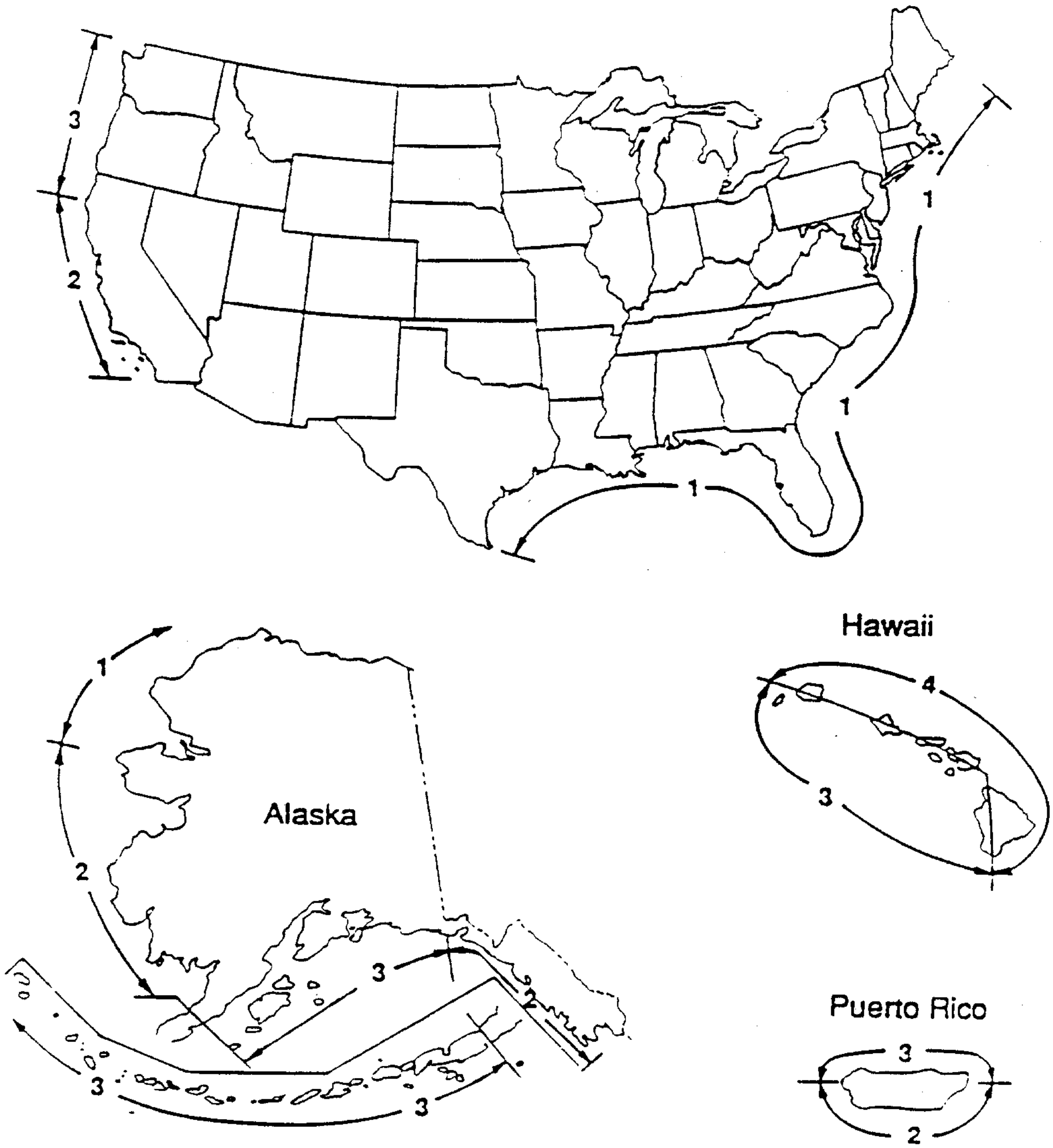
(4) Flooding due to failure of water retention structures. The potential for flooding due to the failure of water retention structures can be assumed to be not significant if the facility is located outside of areas that could be subject to inundation. City, county, state, and federal agencies (e.g., U.S. Army Corps of Engineers, U.S. Bureau of Reclamation) should be contacted as needed to ascertain the location of such water retention structures and inundation areas.

F-4. Evaluation Procedures

The following sections describe evaluation procedures for hazards that are not screened out using the procedures in paragraph F-3. An important element in the evaluations is to assess the consequences of the hazard in terms of the significance of the hazard to the structure. Thus, for example, the occurrence of liquefaction may or may not pose a significant risk to a structure depending on whether or not significant ground and structural deformations could occur as a result of liquefaction. The possible conclusions from these evaluations are: (1) a hazard posing a significant risk to structures does not exist; (2) the hazard exists, but further structural evaluation is required to ascertain whether the risk to structures is significant; or (3) the hazard exists, poses a significant risk of damage to a structure and mitigation measures should be considered.

a. Estimated ground motion. When estimates of earthquake ground shaking parameters are required for these evaluations, they should be consistent with MCE ground motions as defined in Chapter 3. The corresponding performance objectives should be collapse prevention for Seismic Use Groups I and II; for Seismic Use Groups IIIH and IIIE, performance objectives should be 2B and 3B, respectively, as defined in Chapter 4. Estimates of the duration of strong shaking should be based on the assumption of the occurrence of maximum earthquakes in the site region.

b. Surface fault rupture. After a site has been evaluated by the screening criteria developed above and (1) either there is insufficient information to rule out a surface fault rupture hazard, or (2) there is seismic, geomorphic, and/or geologic data that suggests active fault(s) might be present at or near



Zone	Wave Height (feet)
1	5
2	15
3	30
4	50

1 foot = 0.3 meters

Figure F-9 Tsunami zone map and wave heights.

the site, the following information is required to refine definition of the hazard:

- $ the location of fault traces (if any) with respect to the site
- $ the timing of most recent slip activity on the fault
- $ the ground rupture characteristics for a design earthquake on the fault (e.g., type of faulting (Figure F-1), amount of slip and distribution into strike-slip and dip-slip components, and width of the zone of ground deformation)

(1) Fault location. There are several steps that can be taken to confirm and define the location of faults. Further assessments will not be required if it can be shown on the basis of the evaluation procedures outlined below that there are no faults passing beneath the site.

(a) Interpretation of aerial photographs. Aerial photographs can be an excellent supplementary resource to geologic and topographic maps of the site and vicinity for identifying faults. Older photographs are particularly useful if they depict the site and/or its environs prior to development activities that would have altered or destroyed landforms that indicate the presence of faults. For many parts of the country, stereo photographic coverage is available as far back as the 1920s or 1930s. Aerial photographs are usually available from several sources including private companies and from various governmental agencies including the U.S. Geological Survey, U.S. Department of Agriculture (Soil Conservation Service), Bureau of Land Management, Forest Service, etc. The USGS maintains the repository for federal photographic resources at its EROS Data Center, Sioux Falls, South Dakota 57198.

(b) Contacting knowledgeable geologists. There probably are geologists/earth scientists familiar with geologic and tectonic conditions in the site vicinity who will be willing to share their knowledge. These geologists might work for governmental agencies (federal, state, and local), teach and conduct research at nearby colleges and universities, or practice as consultants.

(c) Ground reconnaissance of site and vicinity. Walkdown of the site and its vicinity should be conducted to observe unusual topographic conditions and to evaluate any geologic relationships visible in cuts, channels or other exposures. Features requiring a field assessment might have been identified previously during the geologic and topographic map review, aerial photographic interpretation, and/or during conversations with geologists.

(d) Subsurface exploration. Faults obscured by overburden soils, site grading, and/or structures can be potentially located by one or more techniques. Geophysical techniques such as seismic refraction surveying provide a remote means of identifying the location of steps in a buried bedrock surface and the juxtaposition of earth materials with different elastic properties. Geophysical surveys require specialized equipment and expertise, and their results may sometimes be difficult to interpret. Trenching investigations are commonly used to expose subsurface conditions to a depth of 4.6 to 6.1 m (15 to 20 feet). While expensive, trenches have the potential to locate faults precisely and provide exposures for assessing their slip geometry and slip history. Borings can also be used to assess the nature of subsurface materials and to identify discontinuities in material type or elevation that might indicate the presence of faults.

(2) Fault activity. If it is determined that faults pass beneath the site, it is essential to assess their activity by determining the timing of the most recent slip(s). If it is determined, based on the procedures outlined below, that the faults are not active faults (see paragraph F-3a), then further assessments are not required.

(a) Assess fault relationship to young deposits/surfaces. The most definitive assessment of the recency of fault slip can be made in natural or artificial exposures of the fault where it is in contact with earth materials and/or surfaces of Quaternary age (last 1.8 million years). Deposits might include native soils, glacial sediments like till and loess, alluvium, colluvium, beach and dune sands, and other poorly consolidated surficial materials. Surfaces might include marine, lake, and stream terraces, and other erosional and depositional surfaces. A variety of age-dating techniques, including radiocarbon analysis and soil profile development, can be used to estimate the timing of most recent fault slip.

(b) Evaluate local seismicity. If stratigraphic data are not available for assessment of fault activity, historical seismicity patterns might provide useful information. Maps and up-to-date plots depicting historical seismicity surrounding the site and vicinity can be obtained from the USGS at its National Earthquake Information Center in Golden, Colorado. Additional seismicity information may be obtained from state geologic agencies and from colleges and universities that maintain a network of seismographs (e.g., California Institute of Technology; University of California, Berkeley; University of Nevada, Reno; University of Washington; National Center for Earthquake Engineering Research, Buffalo, New York; etc.). If the fault(s) that pass beneath the site are spatially associated with historical seismicity, and particularly if the seismicity and fault trends are coincident, the faults should probably be considered active.

(c) Evaluate structural relationships. In the absence of both stratigraphic and seismological data, an assessment of the geometric/structural relationships between fault(s) at the site and faults of known activity in the region could be useful. Although less definitive than the two prior criteria, the probability that the site fault is active increases if it is structurally associated with another active fault, and if it is favorably oriented relative to stresses in the current tectonic environment.

(3) Fault rupture characteristics. If the evaluation indicates one or more active faults are present beneath the site, the characteristics of future slip on the fault(s) can be estimated. Based on analysis of moderate and large magnitude earthquakes worldwide, Wells and Coppersmith (1994) have developed empirical relationships among earthquake moment magnitude and a variety of fault characteristics including maximum displacement (Figure F-10) that are based on fault type (e.g., strike-slip, reverse, and normal). These curves provide a convenient means for assessing the amount of slip or displacement fault. Amounts of fault displacement should be estimated assuming the occurrence of a maximum earthquake on the fault. Predicting the width of the zone of surface deformation associated with a surface faulting event is more difficult because empirical relationships having general applicability have not yet been developed. The best means for assessing the width of faulting at the site is site-specific trenching that crosses the entire zone. In the absence of such information, the historical record indicates that steeply dipping faults, such as vertical strike-slip faults, tend to have narrower zones of surface deformation than shallow dipping faults like thrust and normal faults. An example of an evaluation of the potential for surface fault rupture following a screening process is given in Appendix G.

c. Soil liquefaction. If a site has been filtered through the screening criteria and liquefaction-susceptible materials are identified, the potential for liquefaction to occur due to earthquake ground shaking may be assessed by a variety of available approaches (National Research Council, 1985). The most commonly utilized approach is the Seed-Idriss simplified empirical procedure presented by Seed and Idriss (1971, 1982), as updated by Seed et al. (1985) and Youd and Idriss (1997) that utilizes Standard Penetration Test (SPT) blowcount data. The latter citation refers to the Proceedings of the Workshop on Evaluation of Liquefaction Resistance of Soils conducted by the National Center for Earthquake Engineering Research (NCEER). The purpose of the workshop was to update and augment the simplified liquefaction evaluation procedures. Where consensus has been achieved by the workshop participants on changes and additions to the evaluation procedures, these changes and additions are incorporated herein. However, as of October 1998, workshop participants are continuing to evaluate several aspects of the evaluation procedures.

The following paragraphs briefly summarize simplified state-of-the-art approaches for evaluating liquefaction potential and its consequences. Guidance for liquefaction potential evaluations is also presented in Navy Technical Report TR-2077-SHR (Ferritto, 1997b) and Department of Defense Handbook MIL-HDBK-1007/3 (Department of Defense, 1997). Ferritto (1997b) also presents guidance for safety factors against liquefaction and allowable displacements for different facility types.

(1) Seed-Idriss evaluation procedure. Peak ground-surface acceleration, earthquake magnitude, total and effective overburden stresses at the point of interest, and the standardized SPT blowcount are needed to perform the evaluation using the Seed-Idriss simplified empirical procedure. The standardized blowcount index used in the method is $(N_1)_{60}$, which represents the SPT blowcount to advance a 51-mm (2-inch) O.D. split-spoon sampler

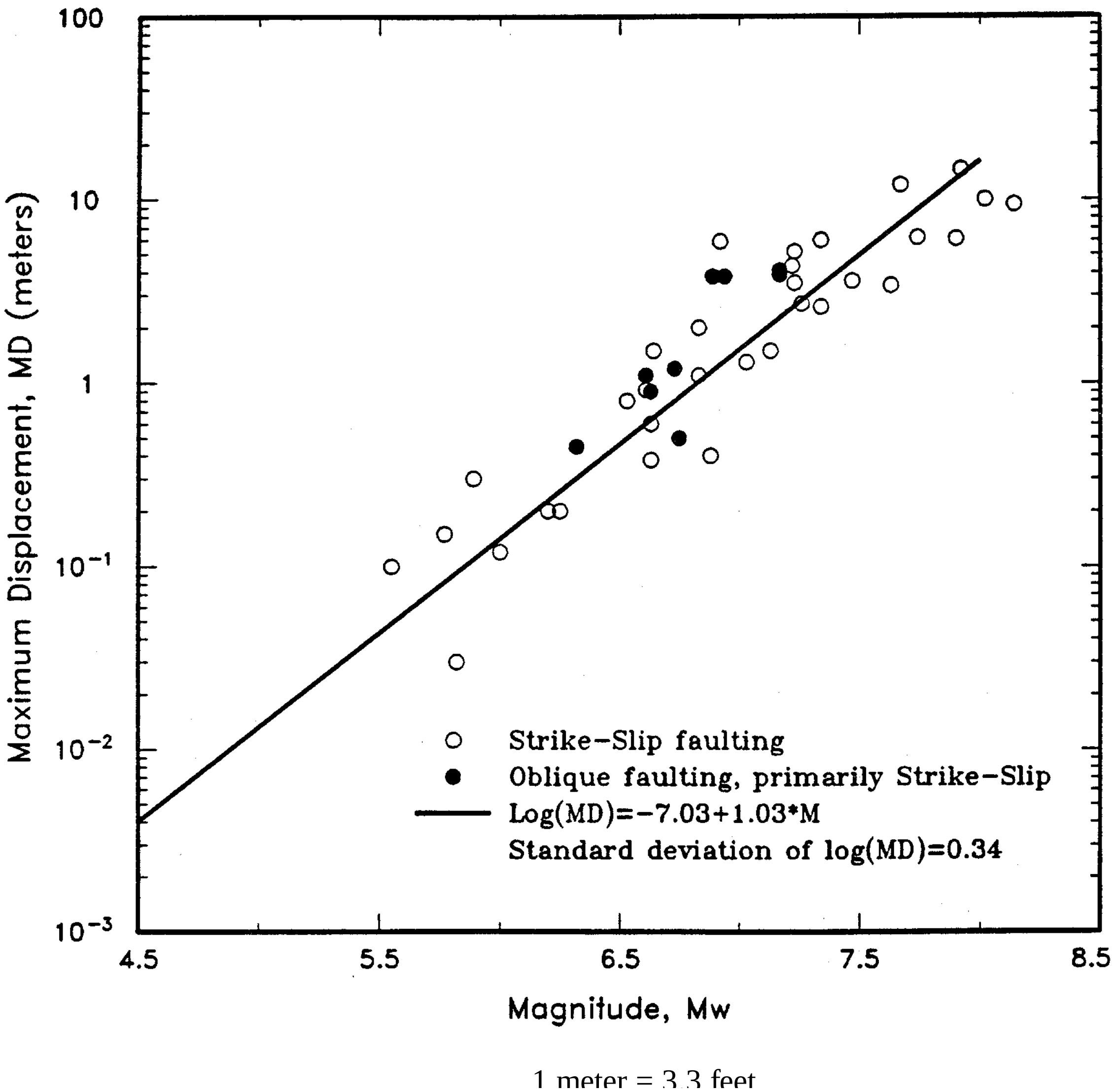


Figure F-10 Relationship between maximum surface fault displacement (MD) and earthquake moment magnitude, M_w, for strike-slip faulting (based on Wells and Coppersmith, 1994).

0.3 m (1 foot) at a 60 percent hammer energy efficiency, with correction to an effective overburden pressure of 96 kPa (2 ksf). The procedure is based on the empirical correlation between cyclic stress ratio (computed from the peak ground surface acceleration) and $(N_1)_{60}$ blow count that differentiates the observed occurrence or non-occurrence of liquefaction in sand deposits during earthquakes. The basic correlation presented by Seed et al. (1985) for magnitude 7.5 earthquakes for materials with different fines contents (FC), and adjusted in Youd and Idriss (1997) for very low blowcounts, is illustrated in Figure F-11; the correlation may be adjusted to other earthquake magnitudes using adjustment factors developed by Seed and Idriss (1982) given in Table F-2. Youd and Idriss (1997) present several alternative magnitude scaling factors; however, at present, consensus has not been attained on revisions to these factors.

(a) For a given value of peak ground surface acceleration (PGA) (in g units) and the total and effective overburden pressures at the depth of interest (s_o and s'_o, respectively), a value of the average induced cyclic stress ratio (CSR) can be computed using the expression (Seed and Idriss, 1971):

$$CSR = \frac{t_a}{s'_o} = 0.65 \frac{PGA}{g} \frac{s_o}{s'_o} r_d$$

in which t_a is the induced average cyclic shear stress at the depth of interest, and r_d is a stress reduction factor that decreases from a value of 1 at the ground surface to a value of 0.9 at a depth of about 10.7 m (35 feet). It is noted that the participants in the NCEER workshop (Youd and Idriss, 1997) have not achieved consensus regarding possible changes to the values for r_d. The relationship for r_d developed by Seed and Idriss (1971) and still in engineering usage is shown in the liquefaction potential evaluation example in Appendix G (Figure G-7). Using values of cyclic stress ratio from the preceding equation and a plot such as Figure F-11 for the appropriate earthquake magnitude, a critical value of the $(N_1)_{60}$ blowcount can be determined, such that those $(N_1)_{60}$ blowcounts exceeding the critical $(N_1)_{60}$ would likely not liquefy and those having a value less than the critical $(N_1)_{60}$ would likely liquefy. By comparing the critical blowcount $(N_1)_{60}$ with the measured $(N_1)_{60}$ of the material, it is possible to assess whether liquefaction would be expected to occur or not at the site. The critical blowcount $(N_1)_{60}$ condition corresponds to a factor of safety against liquefaction equal to unity (i.e., 1.0). Factor of safety is defined as the ratio of the ground-shaking induced cyclic stress ratio (from the preceding equation) to the cyclic resistance ratio (CRR) (see Figure F-11) that defines the boundary between liquefaction and non-liquefaction behavior.

To facilitate the use of electronic computational aids, Youd and Idriss (1997) present equations that may be used to approximate the CRR curves given in Figure F-11. The clean sand curve (fines content < 5 %) is approximated by the following equation:

$$CRR_{7.5} = \frac{a + cx + ex^2 + gx^3}{1 + bx + dx^2 + fx^3 + hx^4} \quad \text{for x < 30}$$

where:

a = 0.048
b = -0.1248
c = -0.004721
d = 0.009578
e = 0.0006136
f = -0.0003285
g = -0.00001673
h = 0.000003714
x = $(N_1)_{60\ cs}$

The curves for silty sands in Figure F-11 may be approximated by correcting the penetration resistance of a silty sand to an equivalent clean sand penetration resistance, $(N_1)_{60cs}$. The equivalent clean sand blowcount may then be used in the preceding equation to estimate liquefaction resistance. The equivalent clean sand blowcount is approximated by the following equation:

$$(N_1)_{60cs} = a + b(N_1)_{60}$$

where:

a	=	0	for FC#5%
a	=	exp[1.76-(190/FC^2)]	for 5%<FC<35%
a	=	5.0	for FC$35%
b	=	1.0	for FC#5%
b	=	[0.99+(FC$^{1.5}$/1000)]	for 5%<FC<35%
b	=	1.2	for FC$35%

where FC is the fines content (expressed as a percentage) measured from laboratory gradation tests from retrieved soil samples.

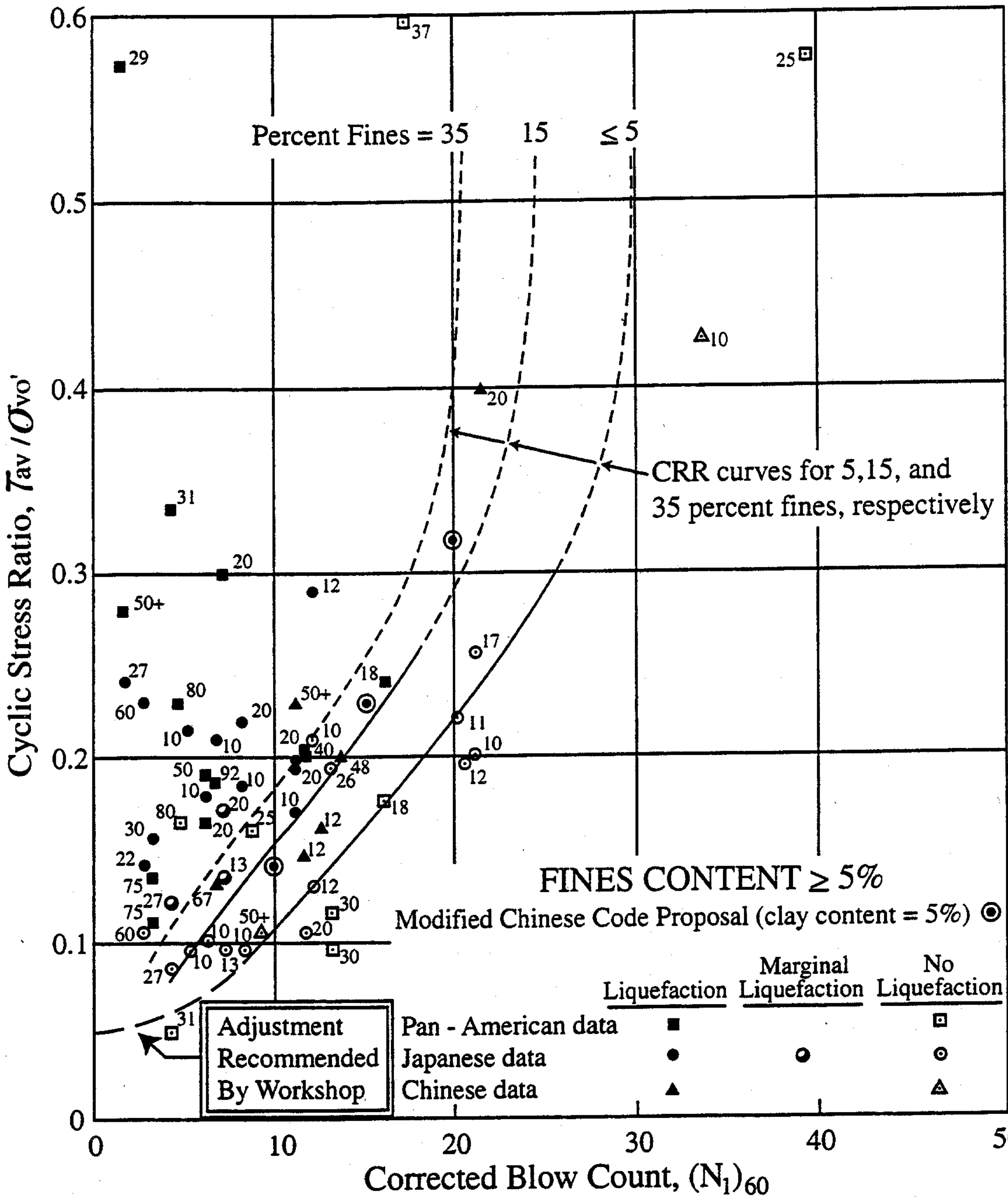


Figure F-11 Relationship between cyclic stress ratio (CSR) causing liquefaction and $(N_1)_{60}$ values for $M_w = 7.5$ earthquakes (Seed et al., 1985; Youd and Idriss, 1997).

Table F-2 Scaling factors for influence of earthquake magnitude on liquefaction resistance (from Seed et al., 1985).

Earthquake Magnitude	**Magnitude Scaling Factor**
M_w	K_m
8½	0.89
7½	1.00
6:	1.13
6	1.32
53	1.50

Note: scaling factors are applied to the ordinates of the curves in Figure F-11.

An example of liquefaction potential evaluation using the simplified empirical procedure is presented in Appendix G. The Navy has developed a computer program, LIQUFAC, for analyzing liquefaction potential using the Seed-Idriss simplified procedure (Ferritto, 1997b). Figure F-12 is a graphic plot illustrating results of LIQUFAC analysis for a soil profile.

(2) Cone Penetration Test (CPT) data are also utilized with the Seed-Idriss evaluation procedure by conversion of the CPT data to equivalent SPT blowcounts, using correlations developed among cone tip resistance (Q_c), friction ratio, soil type, and Q_c/N in which N is the SPT blowcount (e.g., Seed and DeAlba, 1986; Robertson and Campanella, 1985). Direct correlations of CPT data with liquefaction potential have also been developed. The most recent of these are those by Robertson and Wride (1997) and Olsen (1997) in the proceedings of the 1997 NCEER workshop (Youd and Idriss, 1997). To date these are not as widely used as the Seed-Idriss correlation with $(N_1)_{60}$ blowcount in Figure F-11.

(3) Shear wave velocity data have also been correlated with liquefaction potential in a manner similar to the correlations with SPT and CPT data. A recent correlation is presented by Andrus and Stokoe (1997) in the proceedings of the 1997 NCEER workshop (Youd and Idriss, 1997).

(4) Other approaches. The Becker hammer is a larger-diameter penetrometer that has been used to obtain penetration test data in gravelly soils. These data are then correlated to SPT measurements so that liquefaction potential of gravelly soils can be evaluated using Figure F-11. The approach is described by Harder (1997). The threshold strain approach of Dobry et al. (1981) utilizes shear wave velocity as a parameter to estimate a level of cyclic shear strain below which excess pore water pressure will not be generated and accumulated. If the cyclic shear strains induced by an earthquake's ground shaking do not exceed the threshold level, liquefaction cannot occur during that earthquake. National Research Council (1985) notes that this is a conservative evaluation because liquefaction may not occur even if the strains do exceed the threshold.

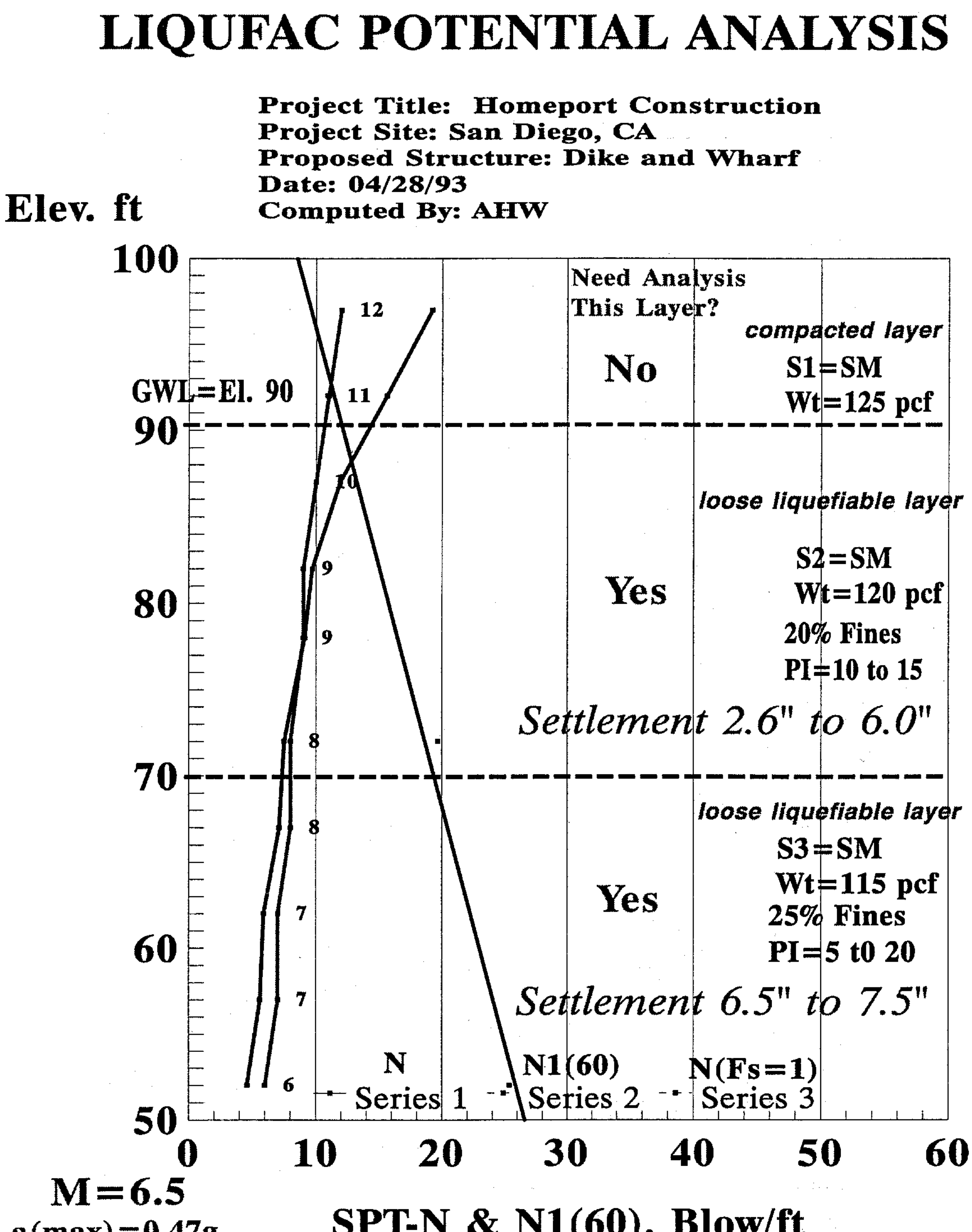
(5) Consequences of liquefaction -- general. The predicted occurrence of liquefaction does not necessarily imply unacceptable adverse consequences to a structure. If liquefaction is estimated to occur under design ground motion levels, the consequences should be assessed. Deformations accompanying liquefaction may or may not be tolerable depending on the specific structure design and performance objectives. Guidance for allowable displacements due to liquefaction for different types of Navy facilities is presented by Ferritto (1997b). Guidelines for assessing consequences of liquefaction are presented in the following paragraphs.

(6) Consequences of liquefaction -- lateral spreads. Lateral spreads are ground-failure phenomena that can occur on gently sloping ground underlain by liquefied soil. Earthquake ground-shaking affects the stability of sloping ground containing liquefiable materials by seismic inertia forces within the slope and by shaking-induced strength reductions in the liquefiable materials. Temporary instability due to seismic inertia forces are manifested by lateral "downslope" movement that can potentially involve large land areas. For the duration of ground shaking associated with moderate-to large-magnitude earthquakes, there could be many such occurrences of temporary instability, producing an accumulation of "downslope" movement.

(a) Various relationships for estimating lateral spreading displacement have been proposed, including the Liquefaction Severity Index (LSI) by Youd and Perkins (1978), a relationship incorporating slope and liquefied soil thickness by Hamada et al. (1986), a modified LSI approach presented by Baziar et al. (1992), and a relationship by Bartlett and Youd (1992, 1995), in which they characterize displacement potential as a function of earthquake and local site characteristics (e.g., ground slope, liquefiable layer thickness, and soil grain size distribution). Equations given by Bartlett and Youd (1992, 1995) for lateral spreading of sloping ground and free-face conditions are as follows:

for free-face conditions:

$$\begin{aligned} LOG(D_H+0.01) = & -16.366 + 1.178\ M \\ & - 0.927\ LOG\ R - 0.013\ R + 0.657\ LOG\ W \\ & + 0.348\ LOG\ T_{15} + 4.527\ LOG\ (100-F_{15}) \\ & - 0.922\ D50_{15} \end{aligned}$$



1 foot = 0.3 meters; 1 pcf = 0.16 kN/m^3

Figure F-12 Example of LIQUFAC analysis graphic plot (Department of Defense, 1997).

and for sloping ground conditions:

$$LOG(D_H+0.01) = -15.787 + 1.178\ M$$
$$-0.927\ LOG\ R - 0.013\ R + 0.429\ LOG\ S$$
$$+ 0.348\ LOG\ T_{15} + 4.527\ LOG\ (100-F_{15})$$
$$- 0.922\ D50_{15}$$

in which:

D_H	=	Displacement (m)
M	=	Earthquake moment magnitude
R	=	Horizontal distance from the seismic energy source, (km).
W	=	100 x (height (H) of the free face / distance (L) from the free face).
S	=	Ground slope (%).
T_{15}	=	Cumulative thickness of saturated granular layers with $(N_1)_{60} \leq 15$, (m).
F_{15}	=	Average fines content of saturated granular layers included in T_{15}, (%).
$D50_{15}$	=	Average mean grain size in layers included in T_{15}, (mm).

(b) This set of relationships is considered to be adequate for most applications to obtain an order of magnitude (i.e., generally within a factor of 2) of the lateral spreading hazard for a site. More site-specific relationships may be developed based on slope stability and deformation analysis for lateral spreading conditions using undrained residual strengths for liquefied sand (Seed and Harder, 1990; Stark and Mesri, 1992) along with simplified Newmark-type (1965) and Makdisi and Seed (1978) displacement approaches, or using more detailed displacement analysis approaches.

(7) Consequences of liquefaction -- flow slides. Flow slides generally occur in liquefied materials located on steeper slopes and may involve ground movements of hundreds of meters. As a result, flow slides can be the most catastrophic of the liquefaction-related ground-failure phenomena. Fortunately, flow slides are much less common occurrences than lateral spreads. Whereas lateral spreading requires earthquake inertia forces to create instability for movement to occur, flow movements occur when the gravitational forces acting on a ground slope exceed the strength of the liquefied materials within the slope.

(8) Consequences of liquefaction -- settlement. With time following the occurrence of liquefaction, the excess pore water pressures built up in the soil will dissipate, drainage will occur, and consolidation or compaction of the soil will occur that will be manifested at the ground surface as settlement. An approach to estimate the magnitude of such ground settlement that is analogous to the simplified empirical procedure for liquefaction potential evaluation (i.e., using SPT blowcount data and cyclic stress ratio) has been presented by Tokimatsu and Seed (1987) and is suggested herein to the user. The relationships presented by Tokimatsu and Seed (1987) are shown on Figure F-13. An example illustrating the estimation of liquefaction-related ground settlement using the Tokimatsu and Seed (1987) procedure is provided in Appendix G. Relationships presented by Ishihara and Yoshimine (1992) are also available for assessing settlement.

(9) Consequences of liquefaction -- bearing capacity reduction. Shaking-induced strength reductions in liquefiable materials that are associated with the generation and accumulation of excess pore water pressure can have effects on the support capacity of foundation elements. For spread-type footings, these effects may be substantial where the groundwater and liquefiable materials are situated at shallow depths relative to the size of the footing and when liquefaction or high levels of excess pore water pressure occur (i.e., when the factor of safety against liquefaction is less than about 1.5; see, for example Figure 27 of Marcuson et al., 1990). Figure F-14 illustrates the relative effects that high excess pore water pressure or liquefaction may have on the calculated ultimate bearing capacity of a spread footing. The effects illustrated in Figure F-14 were developed considering representative density and strength properties for non-liquefied soil (i.e., friction angle) and liquefied soil (i.e., undrained residual strength [e.g., Seed and Harder, 1990; Stark and Mesri, 1992]), the Marcuson et al. (1990) relationship between excess pore water pressure and factor of safety against liquefaction, and static ultimate bearing capacity formulations for layered systems (e.g., Meyerhof, 1974; Hanna and Meyerhof, 1980; Hanna, 1981). Meyerhof (1974) and Hanna and Meyerhof (1980) address footings in sand overlying clay, which can be used for evaluation of a liquefaction condition, treating the liquefied material as a clay with strength characterized by undrained residual strength, whereas Hanna (1981) addresses footings in strong sand overlying weak sand, which can be used for either liquefaction or high excess pore pressure.

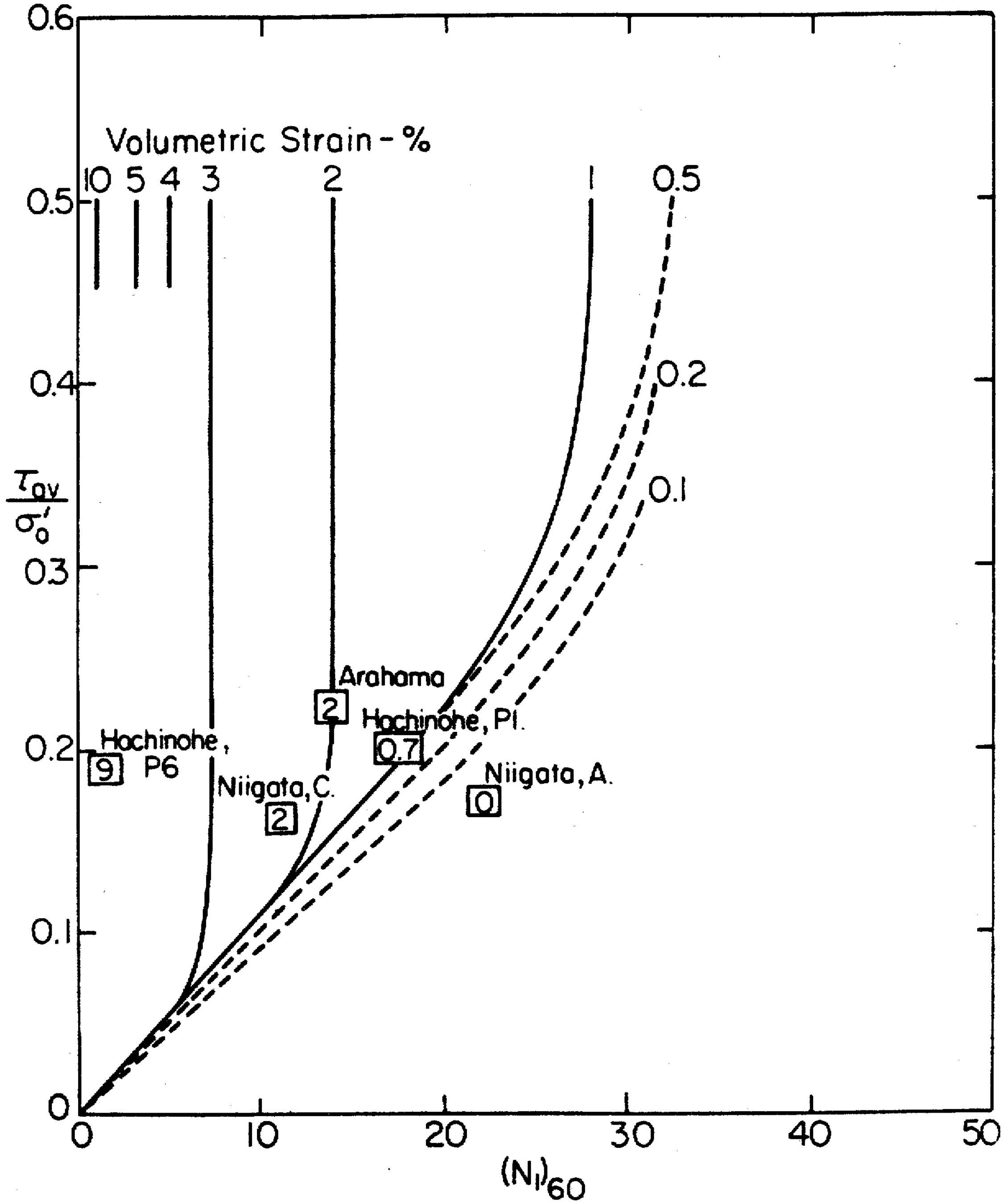


Figure F-13 Relationship between cyclic stress ratio (CSR), $(N_1)_{60}$, and volumetric strain for saturated clean sands (from Tokimatsu and Seed, 1987).

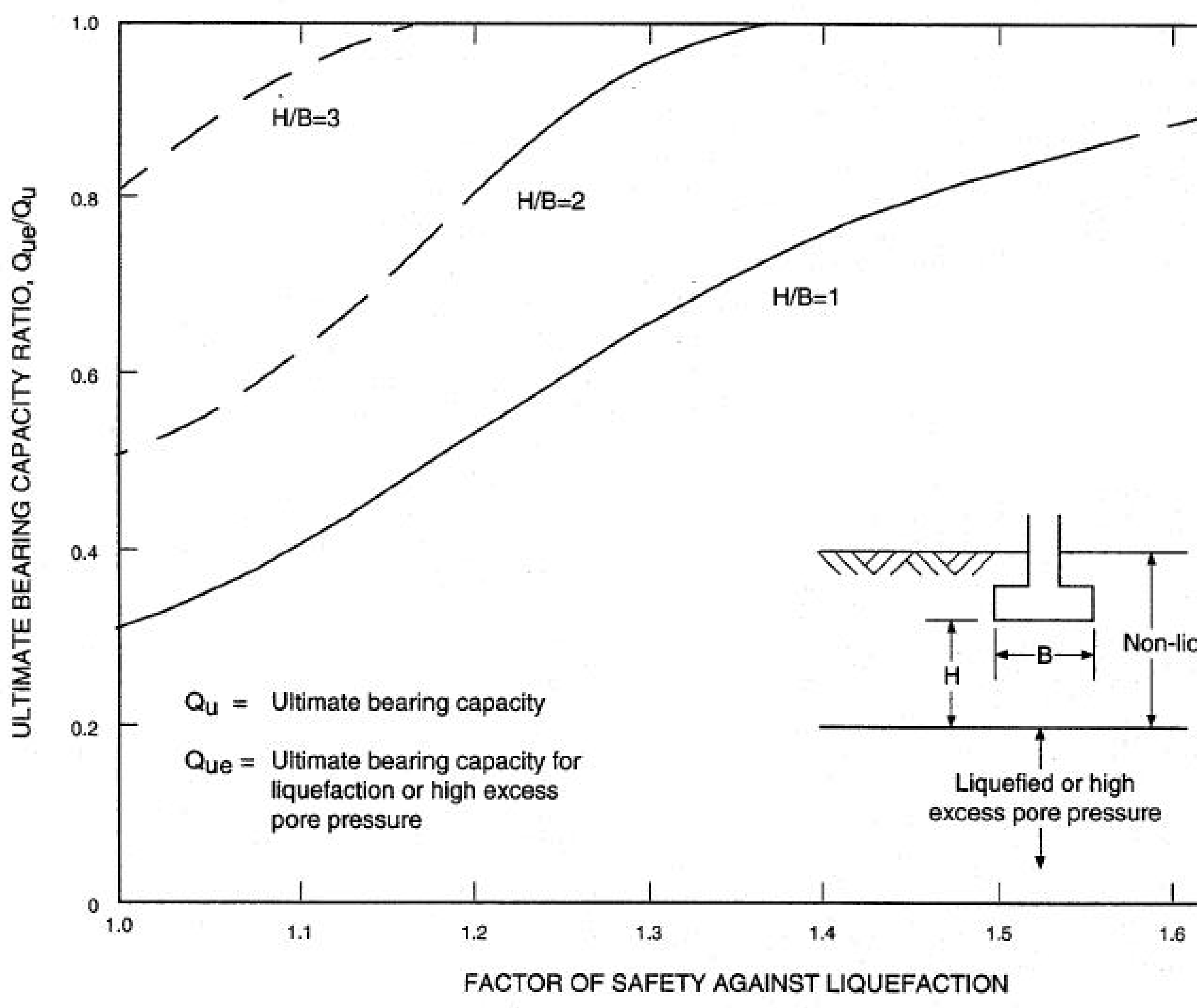


Figure F-14 Illustration of effects of liquefaction or increased pore wat er pressures on ultimate bearing capa
foundations.

(a) Richards et al. (1993) suggest that, in addition to strength reductions accompanying high excess pore water pressures and liquefaction, lateral inertial forces in the soil may reduce the bearing capacity of a shallow foundation system, thereby affecting the settlement performance of the foundation. However, the importance of this phenomenon in comparison to the geologic hazards addressed in this appendix is not yet clearly demonstrated by case histories. The phenomenon should be considered when evaluating foundation bearing capacities as part of a seismic rehabilitation design process.

(10) Consequences of liquefaction -- increased lateral pressures on walls. Behind a wall, the buildup of pore water pressures during the liquefaction process increases the pressure on the wall. This pressure is a static pressure which reduces with time after the earthquake as pore pressures dissipate. Ebeling and Morrison (1992) provide procedures for assessing effects of variable amounts of pore pressure buildup on the lateral pressures behind walls. In addition, the Ebeling and Morrison (1992) procedures cover the transient, dynamic pressures on walls induced by earthquake ground shaking. Both types of increases in lateral pressures due to earthquakes may influence the behavior of retaining walls, although most cases of retaining wall failures during earthquakes have been associated with liquefaction of loose sand backfills behind waterfront retaining walls. Department of Defense (1997) presents design procedures for steel sheet pile walls based on the procedures developed by Ebeling and Morrison (1992).

(11) Consequences of liquefaction -- flotation of buried structures. The potential for flotation of a buried or embedded structure can be evaluated by comparing the total weight of the buried or embedded structure with the increased uplift forces occurring due to the buildup of liquefaction-induced pore water pressures.

d. Differential compaction. The procedures described by Tokimatsu and Seed (1987) are suggested for estimating earthquake-induced settlements due to densification of saturated and unsaturated cohesionless soils. Other procedures can be used if justified. The principal soil parameter required for evaluations using the Tokimatsu and Seed (1987) method is the normalized Standard Penetration Test (SPT) resistance, $(N_1)_{60}$, in blows/foot. Appendix G provides an example of the application of this methodology. It is noted that the procedure provides an estimate of the total earthquake-induced settlement at a site for a given soil profile. The differential settlement must then be assessed based on considerations of soil variability and other factors.

e. Landsliding. Prior to performing engineering analyses to assess landslide potential, the data gathered in the screening stage should be supplemented if necessary. More detailed geologic reconnaissance and mapping may be needed. If preexisting landslides were identified at the site in the screening stage, subsurface investigations may be required to assess the slide geometry. Geotechnical data should be reviewed to assess the engineering properties of the subsurface materials in the slope(s). If sufficient data are lacking, supplemental field and laboratory testing may be required. For slopes located in stiff, nonsensitive clays, dry sands, and saturated sands that do not liquefy or lose their strength during earthquake shaking, the stability of the slopes can be evaluated using either pseudo-static analysis or deformation analysis procedures. The deformation behavior of slopes that liquefy is addressed in paragraph F-4c.

(1) Pseudo-static analysis procedure. The pseudo-static analysis can be used in the initial evaluation. In the pseudo-static analysis, inertial forces generated by the earthquake are represented by an equivalent static horizontal force (seismic-coefficient) acting on the potential sliding mass. In this analysis, the seismic coefficient should be equal to the peak ground acceleration in the vicinity of the slope. The factor of safety for a given seismic coefficient can be estimated using limit equilibrium slope stability methods. A computed factor of safety greater than one indicates that the slope is stable and further evaluations are not required. A computed factor of safety of less than one indicates that the slope will yield and deformations can be expected. In this case, an estimate of the expected slope deformations should be made using the procedures described below.

(2) Deformation analysis procedures. Simplified procedures for estimating deformations of slopes during earthquake shaking are based on the concept of yield acceleration originally proposed by Newmark (1965). Newmark's method has been modified and augmented by several investigators (Goodman and Seed, 1966; Ambraseys, 1973;

Sarma, 1975; Franklin and Chang, 1977; Makdisi and Seed, 1978; Hynes-Griffin and Franklin, 1984; Wilson and Keefer, 1985; Lin and Whitman, 1986, Yegian et al., 1991). The procedure assumes that movement occurs on a well-defined slip surface and that the material behaves elastically at acceleration levels below the yield acceleration but develops a perfectly plastic behavior above yield. The procedure involves the following steps:

- A yield acceleration, k_y, i.e., the acceleration at which a potential sliding surface would develop a factor of safety of unity, is determined using limit equilibrium pseudo static slope stability methods. Values of the yield acceleration are dependent on the slope geometry, groundwater conditions, the undrained shear strength of the slope material (or the reduced strength due to earthquake shaking), and the location of the potential sliding surface.

- The peak or maximum acceleration, k_{max}, induced within a potential sliding mass (average of the peak accelerations over the mass) must be estimated. Often this value is assumed equal to the free field ground surface acceleration, a_{max}. This neglects possible amplification of accelerations on a slope due to topographic effects, but also neglects reduction of acceleration due to reduction of ground motion with depth and averaging over the sliding mass. A specific evaluation of k_{max} considering amplifying and reducing effects can always be made using dynamic response analysis or simplified methods.

- If the maximum induced acceleration, k_{max}, exceeds the yield acceleration, k_y, downslope movement of the sliding mass occurs. Conceptually, if there is a time history of induced accelerations, some of which exceed the yield acceleration, downslope movement occurs when the induced accelerations exceed the yield acceleration. Movement stops after the time when the induced acceleration level drops below the yield acceleration. The magnitude of the potential displacements can be calculated by a simple double integration procedure of an accelerogram (see Figure F-15 for an illustration).

(a) The above procedure was used by Makdisi and Seed (1978) to develop a simplified procedure for estimating displacements in dams and embankments. Charts relating the displacements as a function of the ratio of the yield acceleration to the maximum induced acceleration (k_y/k_{max}) are shown on Figures F-16 and F-17. The displacements shown on Figures F-16 and F-17 are normalized with respect to the amplitude of the peak induced acceleration, k_{max} (expressed as a decimal fraction of gravity), and the predominant period of the induced acceleration time-history, T_o.

(b) A convenient relationship (Egan, 1994) derived from the results of Makdisi and Seed (1978) is shown on Figure F-18. The displacement per cycle of significant shaking normalized with respect to the induced peak acceleration (expressed as a decimal fraction of gravity) is plotted against the ratio of the yield acceleration to the induced peak acceleration. The curves are most representative for ground motions having a predominant period of about one second. Shown on the same figure is a relationship between earthquake magnitude and number of cycles of significant shaking (Seed and Idriss, 1982).

(c) The Newmark sliding block analysis concept was also employed by Franklin and Chang (1977) who computed permanent displacements based on a large number of recorded acceleration time-histories from previous earthquakes and a number of synthetic records. Their results are shown on Figure F-19 in terms of upper bound envelop curves for standardized maximum displacements versus the ratio of the yield acceleration to the maximum earthquake acceleration. The time-histories used by Franklin and Chang (1977) were all scaled to a peak ground acceleration of 0.5g and peak ground velocity of 30 inches per second. The displacement (inches) for particular values of peak ground acceleration, A, and velocity, V, may be obtained by multiplying the standardized maximum displacement by the quantity $V^2/1800A$, where V is in units of inches per second and A is a decimal fraction of gravity.

(d) Yegian et al. (1991) performed similar analyses using 86 ground motion records. Their computed normalized displacements are shown on Figure F-20. Their computed displacements were normalized with respect to the peak-induced

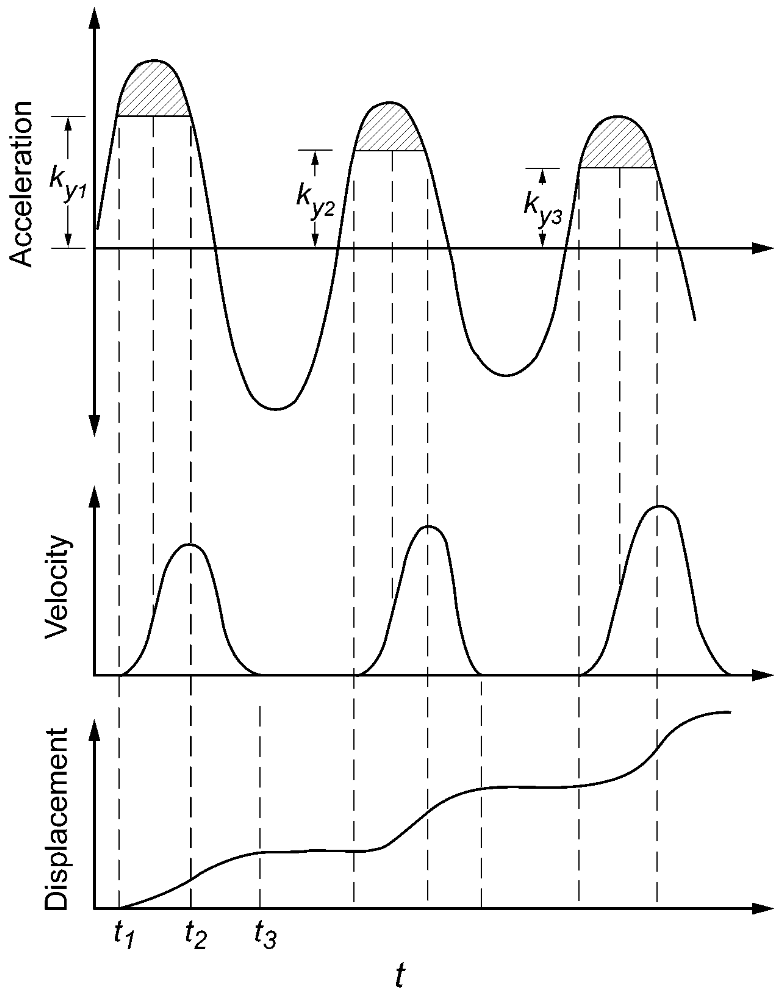


Figure F-15 Integration of acceleration time-history to determine velocities and displacements (from Goodman and Seed, 1966).

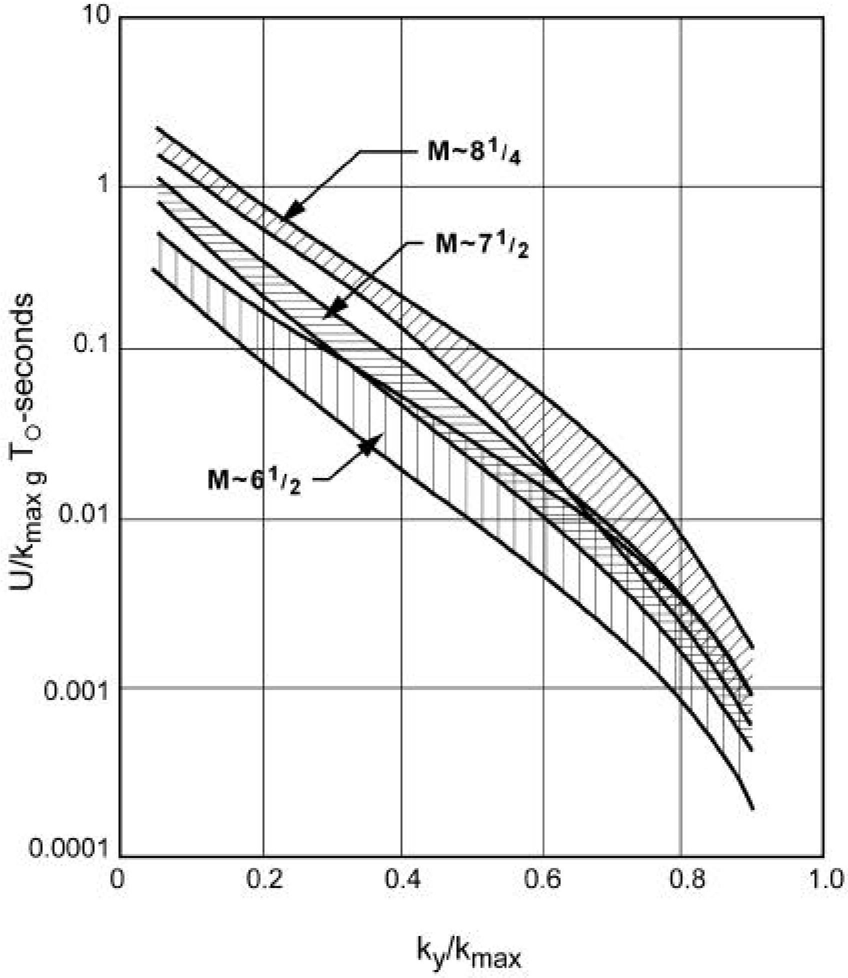


Figure F-16 Variation of normalized permanent displacement with yield acceleration-summary of all data (from Makdisi and Seed, 1978)

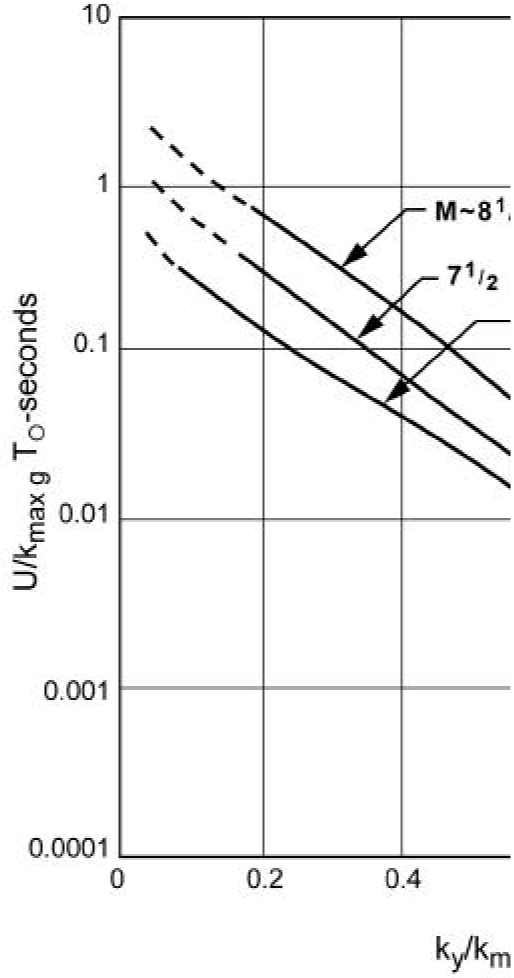


Figure F-17 Variation of averag
displacement with
Makdisi and Seed,

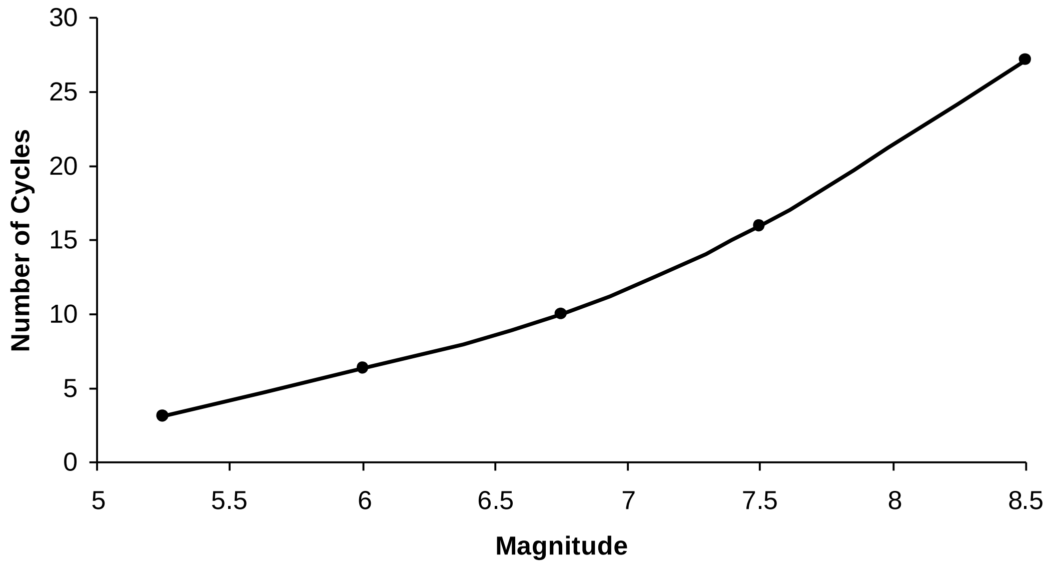


Figure F-18a Relationship between earthquake moment magnitude and number of cycles (after Seed and Idriss, 1982).

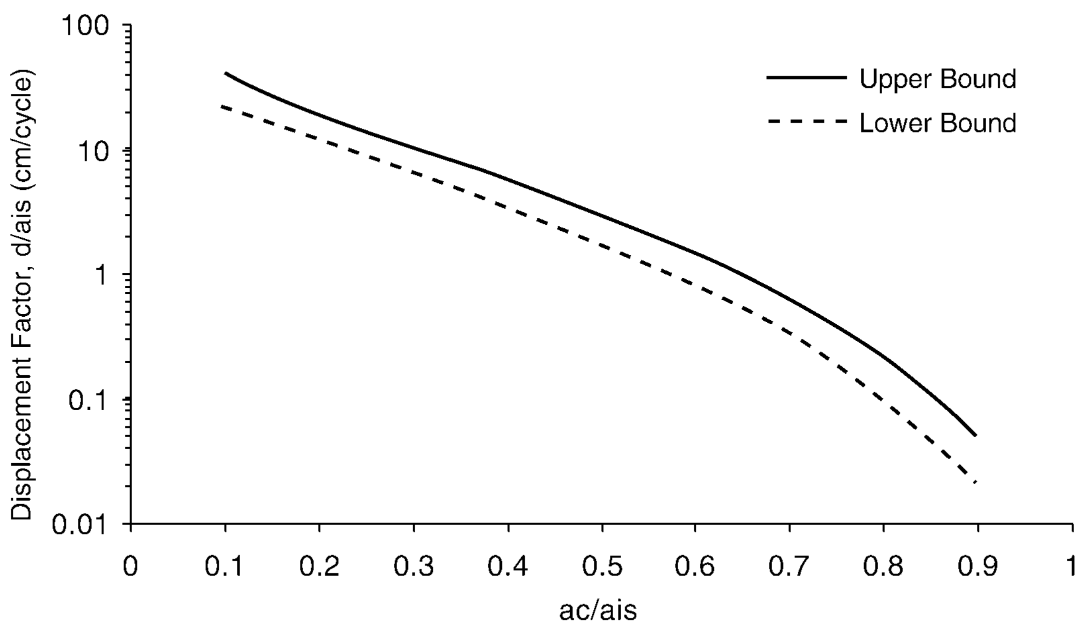
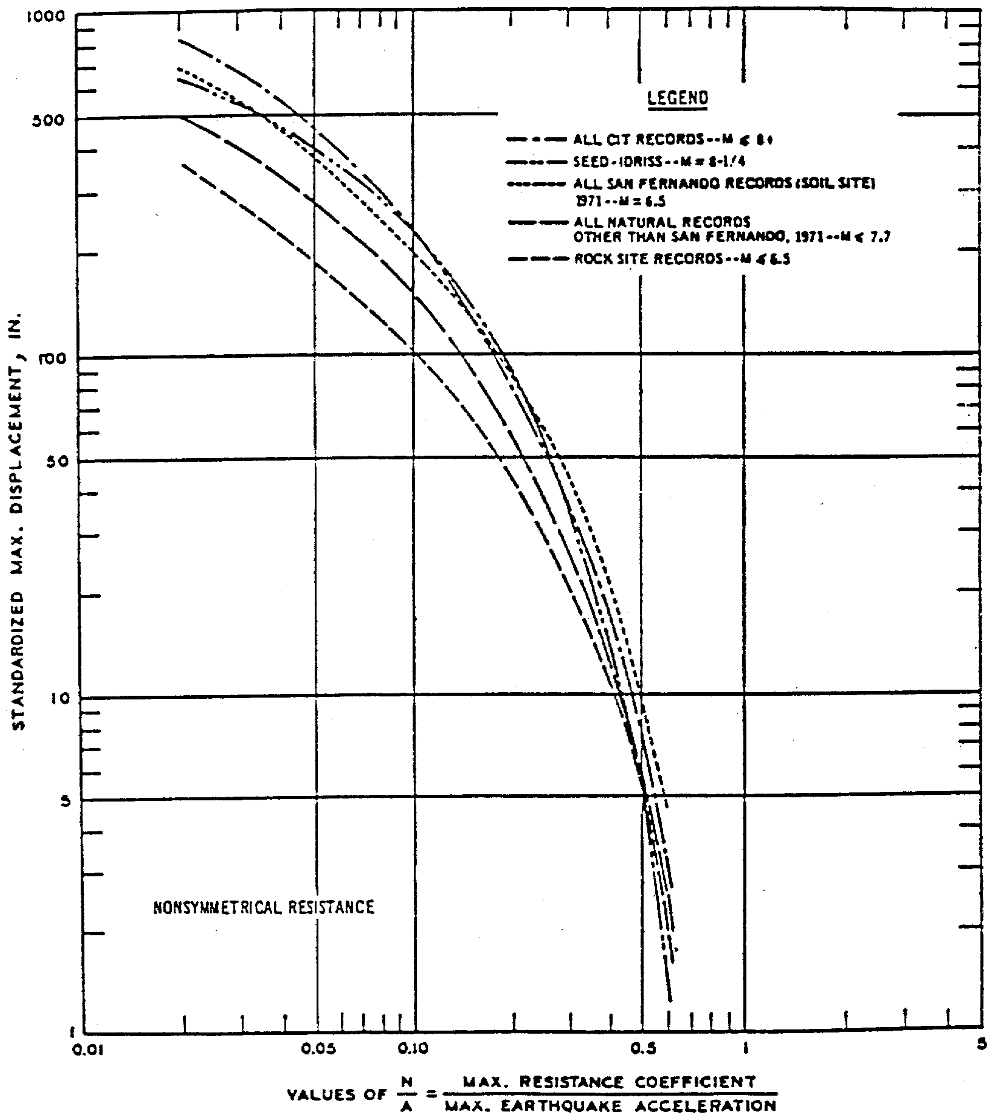


Figure F-18b Relationship between displacement factor and ratio of critical acceleration and induced acceleration (after Egan, 1994).



1 inch = 2.5 cm

Figure F-19 Upper bound envelope curves of permanent displacements for all natural and synthetic records analyzed (from Franklin and Chang, 1977).

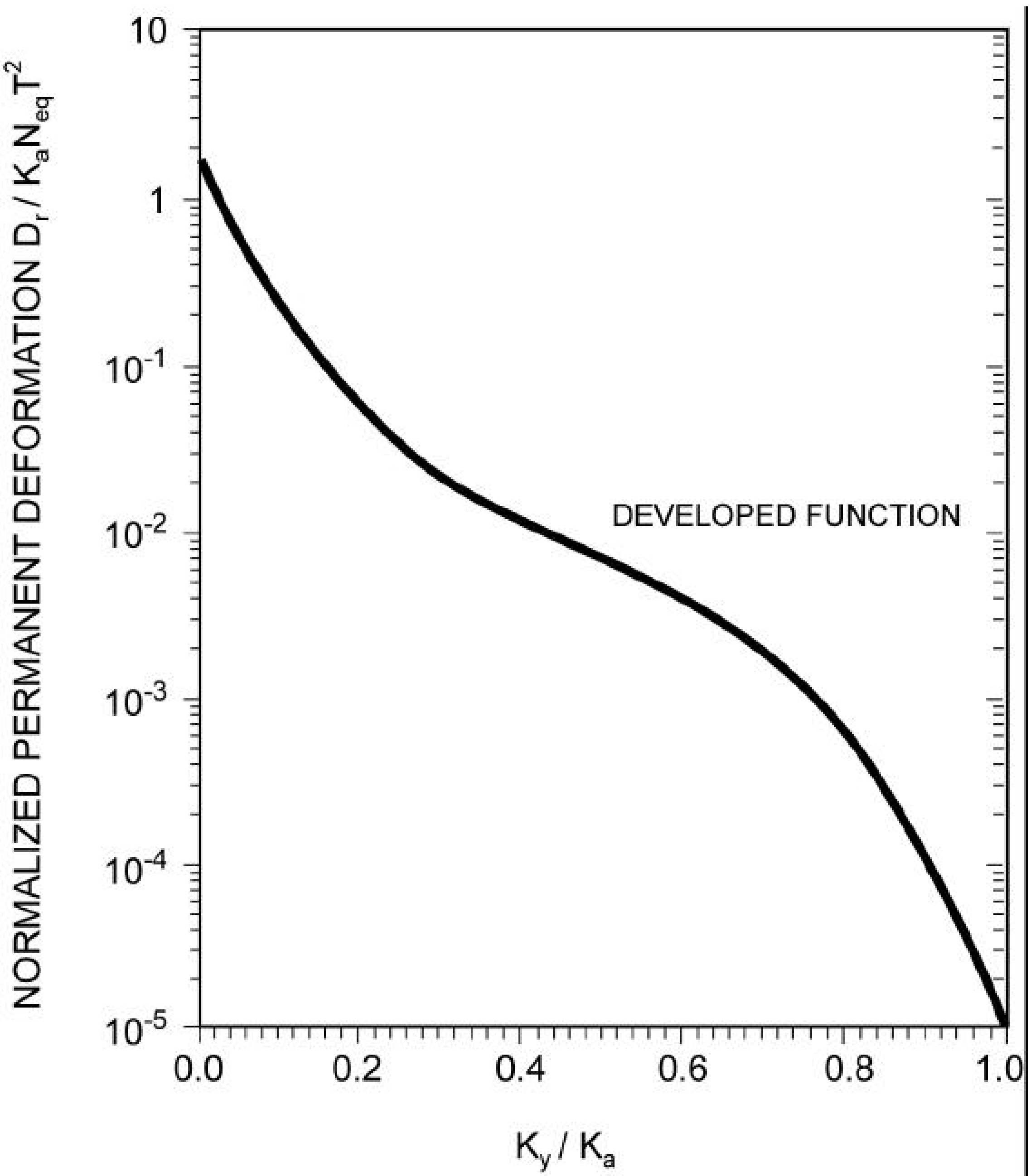


Figure F-20 Variation of normalized permanent deformation with yield acceleration (from Yegian et al., 1991).

acceleration, k_a (units of g), the number of equivalent cycles, N_{eq}, and the square of the natural period of the time-history, T.

(3) Example. An example of a detailed evaluation of landslide potential is given in Appendix G.

f. Flooding. If a facility has possible exposure to earthquake-induced flooding after applying the screening criteria in Section F-3, then further evaluations should be directed at assessing the potential, severity, consequences, and likelihood of the hazard. The evaluation of the potential for landsliding into or within a body of water utilizes methodologies described previously in the section for assessing liquefaction and landsliding. Evaluation of the height of waves that could be produced by a tsunami, seiche, or landslide requires special expertise in fields such as fluid dynamics and coastal engineering as well as seismological, geophysical, and earthquake engineering expertise in characterizing the earthquakes and ground shaking that cause these phenomena. Similarly, geological, seismological, and geophysical expertise are required to assess tectonic movements such as uplift or tilting that could cause flooding. Such studies of hazard potential and severity should be undertaken unless it can be concluded that the effects of flooding on the facility site are tolerable considering the performance objective for the facility, or the probability of occurrence of the hazard is sufficiently low that the risk can be accepted.

(1) If a facility has possible exposure to flooding from failure of a water retention structure, the agencies having jurisdiction over these facilities should be contacted to ascertain whether the structure has been evaluated or designed for appropriate ground shaking using modern seismic analysis and design methods. The potential effects of the flooding at the site should also be evaluated.

F-5. Mitigation Techniques and Considerations

In the event that a significant geologic hazard is found to exist at a facility site, alternatives for mitigating the hazard should be identified and evaluated.

a. Overall approaches to hazard mitigation. The overall approaches to hazard mitigation are (1) eliminating or reducing the hazard; (2) eliminating or reducing the consequences of the hazard; and (3) resiting the proposed facility to a less hazardous location. The following paragraphs summarize hazard mitigation strategies that have been used or considered for the different geologic hazards.

b. Surface fault rupture. There is no mitigation technique that can prevent fault rupture from occurring. Therefore, if the risk posed by the hazard of surface fault rupture is unacceptable, then the mitigation options are either avoiding the hazard by resiting or designing for the displacements.

(1) Generally, it is not feasible to design for the large and concentrated displacements associated with surface fault rupture. However, during the 1978 Managua, Nicaragua earthquake, the foundation and basement of the Banco Central building were apparently rigid and strong enough to divert a fault slippage of several inches around the building and the building sustained only minor damage due to the faulting (Wyllie et al., 1977; Youd, 1989). Thus, the possibility of mitigation by designing for fault displacement should be considered unless the displacements are of a magnitude that obviously would not be tolerable.

c. Soil liquefaction. Ground modification techniques can be considered to eliminate or reduce the liquefaction potential hazard. Soil modification techniques that can be considered include soil removal and replacement, vibratory soil densification, soil grouting, installation of drains, and installation of permanent dewatering systems. A number of ground modification techniques are summarized in Table F-3 (National Research Council, 1985; Ferritto, 1997b).

(1) Soil removal and replacement. Removing liquefiable soil and replacing it with soil that is not liquefiable (including recompaction of the excavated soil in lifts to a dense, nonliquefiable state) is a positive method for mitigating a liquefaction hazard. However, it may not be economically feasible in many cases because of the need to dewater a site to remove the soil as well as the need to retain the area surrounding the site if existing facilities are nearby. The effect of dewatering and excavation on adjacent facilities should also be evaluated.

(2) In-place soil densification. Various techniques can be considered to increase the density of the in-place soil, thereby reducing its tendency to compact and buildup pore pressures during an earthquake. A number of methods are summarized in Table F-3. In-place soil densification is often the

Table F-3 Liquefaction remediation measures (National Research Council, 1985; Ferritto, 1997b).

Method	Principle	Most Suitable Soil Conditions/Types	Maximum Effective Treatment Depth	Relative Cost
1. Blasting	Shock waves and vibrations cause limited liquefaction, displacement, remolding, and settlement to higher density.	Saturated, clean, sands; partly saturated sands and silts after flooding.	>40 m	Low
2. Vibratory probe a. Terraprobe b. Vibrorods c. Vibrowing	Densification by vibration; liquefaction-induced settlement and settlement in dry soil under overburden to produce a higher density.	Saturated or dry clean sand; sand.	20 m routinely (ineffective above 3-4 m depth); >30 m sometimes; vibrowing, 40 m	Moderate
3. Vibrocompaction a. Vibroflot b. Vibro-Composer System	Densification by vibration and compaction of backfill material of sand or gravel.	Cohesionless soils with less than 20% fines.	>30 m	Low to moderate
4. Compaction piles	Densification by displacement of pile volume and by vibration during driving; increase in lateral effective earth pressure.	Loose sandy soil; partly saturated clayey soil; loess.	>20 m	Moderate to high
5. Heavy tamping (dynamic compaction)	Repeated application of high-intensity impacts at surface.	Cohesionless soils best; other types can also be improved.	30 m (possibly deeper)	Low
6. Displacement/ compaction grout	Highly viscous grout acts as radial hydraulic jack when pumped in under high pressure.	All soils.	Unlimited	Low to moderate
7. Surcharge/ buttress	The weight of a surcharge/ buttress increases the liquefaction resistance by increasing the effective confining pressures in the foundation.	Can be placed on any soil surface.	--	Moderate if vertical drains used
8. Drains a. Gravel b. Sand c. Wick d. Wells (for permanent dewatering)	Relief of excess pore water pressure to prevent liquefaction. (Wick drains have comparable permeability to sand drains.) Primarily gravel drains; sand/wick may supplement gravel drain or relieve existing excess pore water pressure. Permanent dewatering with pumps.	Sand, silt, clay.	Gravel and sand >30 m; depth limited by vibratory equipment; wick >45 m	Moderate to high
9. Particulate grouting	Penetration grouting—fill soil pores with soil, cement, and/or clay	Medium to coarse sand and gravel	Unlimited	Lowest of grout methods
10. Chemical grouting	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate.	Medium silts and coarser	Unlimited	High
11. Pressure-injected lime	Penetration grouting—fill soil pores with lime.	Medium to coarse sand and gravel.	Unlimited	Low

Table F-3 Liquefaction remediation measures (National Research Council, 1985; Ferritto, 1997b).

Method	Principle	Most Suitable Soil Conditions/Types	Maximum Effective Treatment Depth	Relative Cost
12. Electrokinetic injection	Stabilizing chemicals move into and fill soil pores by electro-osmosis or colloids into pores by electro-phoresis.	Saturated sands, silts, silty clays.	Unknown	High
13. Jet grouting	High-speed jets at depth excavate, inject, and mix a stabilizer with soil to form columns or panels.	Sands, silts, clays.	Unknown	High
14. Mix-in-place piles and walls	Lime, cement, or asphalt introduced through rotating auger or special in-place mixer.	Sand, silts, clays, all soft or loose inorganic soils.	>20 m (60 m obtained in Japan)	High
15. In-situ vitrification	Melts soil in place to create an obsidian-like vitreous material.	All soils and rock.	>30 m	Moderate
16. Vibro-replacement stone and sand columns a. Grouted b. Not grouted	Hole jetted into fine-grained soil and backfilled with densely compacted gravel or sand hole formed in cohesionless soils by vibro techniques and compaction of backfilled gravel or sand. For grouted columns, voids filled with a grout.	Sands, silts, clays.	>30 m (limited by vibratory equipment)	Moderate
17. Root piles, soil nailing	Small-diameter inclusions used to carry tension, shear, compression.	All soils.	Unknown	Moderate to high

most cost-effective way to mitigate a liquefaction hazard if the densification process can be undertaken without adverse effects on adjacent structures (e.g., potential effects of settle ment or vibration). Figure F-21 illustrates the technique of vibro-replacement in which a vibrating probe is inserted into the soil at close spacings and gravel or crushed rock is also placed at the vibration locations to create a dense gravel column surrounded by densified in-placed soil.

(3) Different types of grouting that can be considered include permeation grouting, compaction grouting, and formation of grouted soil columns. Permeation grouting involves injecting chemical grout into liquefiable sands to essentially replace the pore water and create a non-liquefiable solid material in the grouted zone. The more fine-grained and silty the sands, the less effective is permeation grouting. Compaction grouting involves pumping a mixture of soil, cement, and water into the ground to form bulbs of grouted material. The formation of these bulbs compresses and densifies the surrounding soil, thus reducing its liquefaction potential. However, the amounts of densification that can be achieved may be limited because static compression is less effective than vibration in densifying sands. Compaction grouting must be done carefully to avoid creating unacceptable heaving or lateral displacements of adjacent structures during the grouting process. The mixing or injection of grout locally beneath foundation locations can also be accomplished to form stabilized columns of soil to transfer vertical foundation loads to deeper nonliquefiable strata.

(4) Drain installation (e.g., stone or gravel columns) involves creating closely spaced, vertical columns of permeable material in the liquefiable soil strata. Their purpose is to dissipate soil pore water pressures as they build up during the earthquake shaking, thus preventing liquefaction from occurring. To achieve the objective of high permeability in the gravel column, it must be constructed by a method that avoids contamination by a mixing of the gravel with the surrounding soil. Permanent dewatering systems lower groundwater levels below liquefiable soil strata, thus preventing liquefaction.

(5) All of the above techniques can potentially be applied beneath the building area to prevent the occurrence or reduce the extent and effects of liquefaction. If the assessed consequences of liquefaction are reduction of bearing capacity and/or building settlements, these measures should be sufficient. However, if a potential for significant liquefaction-induced lateral spreading exists at a site, then ground modification beyond the immediate building area may need to be considered. This is because the potential for lateral spreading movements beneath a building may depend on the behavior of the soil mass at distances well beyond the building as well as immediately beneath it. Thus, measures to prevent lateral spreading may, in some cases, require stabilizing large soil volumes and/or constructing buttressing structures that can reduce the potential for or the amount of lateral movements.

(6) Modifications to the structure or its foundation may also be considered to mitigate the consequences. If the predicted movements are small, the structure can be strengthened to resist the deformations. The foundation system can be designed to reduce or eliminate the potential for large foundation displacements, for example by using deep foundations to achieve bearing on a deeper, non-liquefiable strata. Alternatively, a shallow foundation system can be made more rigid (for example by a system of well-reinforced grade beams or mats between isolated footings) in order to reduce the differential ground movements transmitted to the structure.

(7) Conceptual schemes to mitigate liquefaction-induced settlement or bearing capacity reductions are illustrated in Figure F-22. Conceptual schemes to mitigate liquefaction induced lateral spreading are illustrated in Figure F-23. Remediation methodologies are discussed in more detail in a number of publications, including Mitchell (1981), Ledbetter (1985), National Research Council (1985), ASCE (1997), Department of Defense (1997), and Ferritto (1997b), Mitchell et al. (1998).

d. Soil differential compaction. For cases of predicted significant differential settlements of a building, mitigation options are similar to those for mitigating liquefaction potential beneath a building. These options include modifying the soil or groundwater conditions beneath the building, designing the structure to withstand the ground movements, or modifying the foundation system by deepening or stiffening.

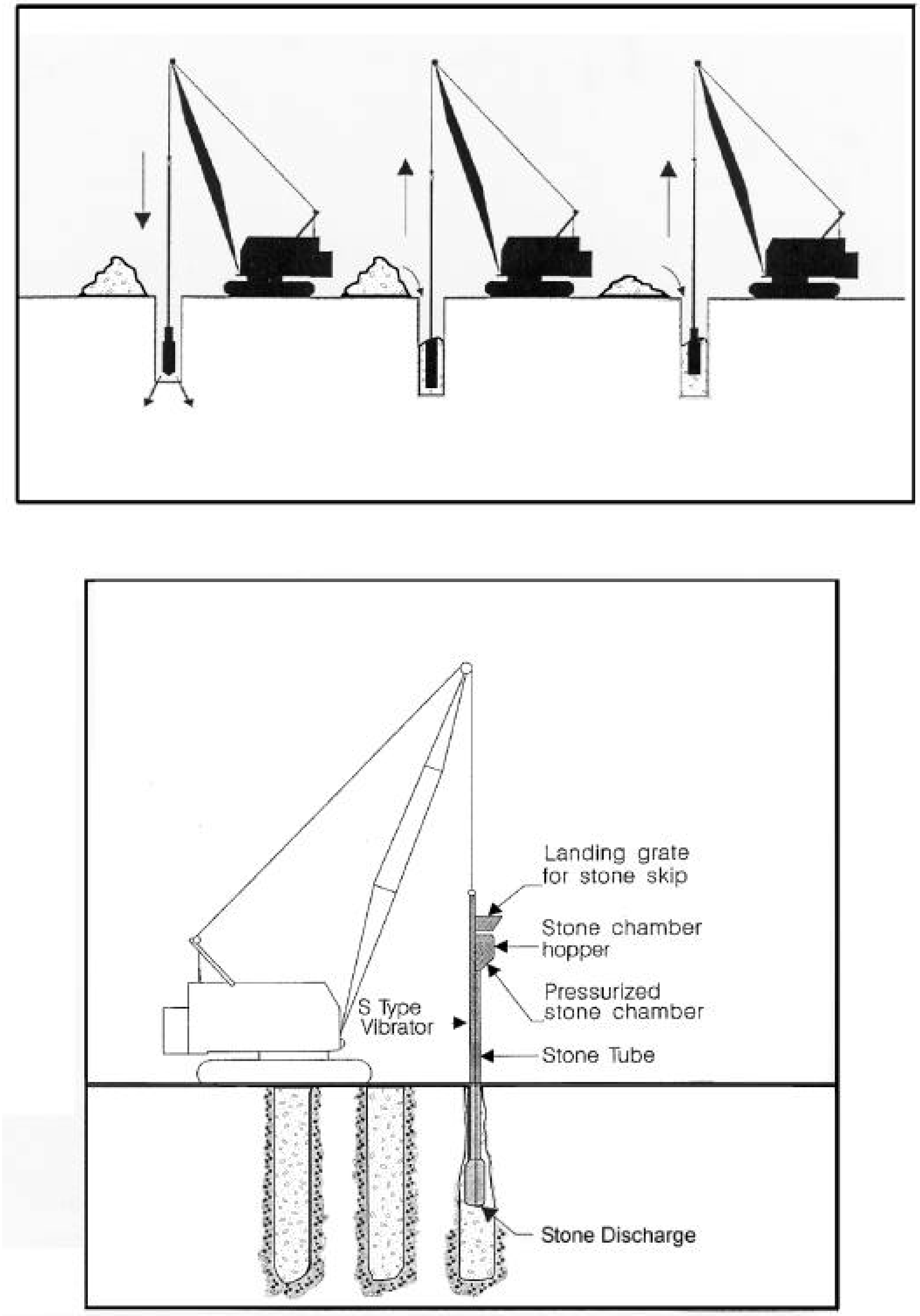


Figure F-21 Vibroreplacement and installation of stone columns (after Baez and Martin, 1992; Department of Defense, 1997).

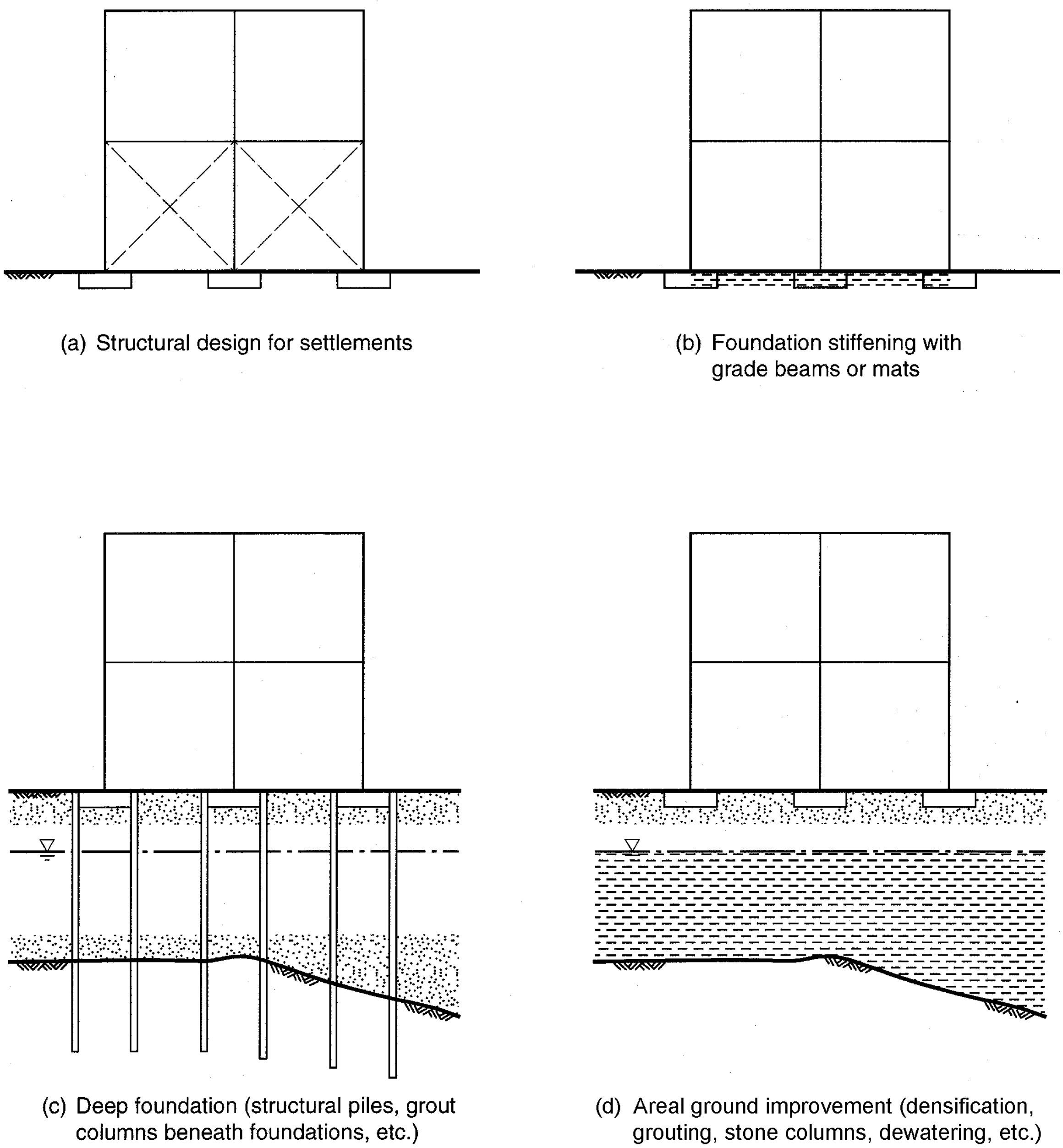


Figure F-22 Conceptual schemes to resist liquefaction-induced settlement or bearing capacity reductions.

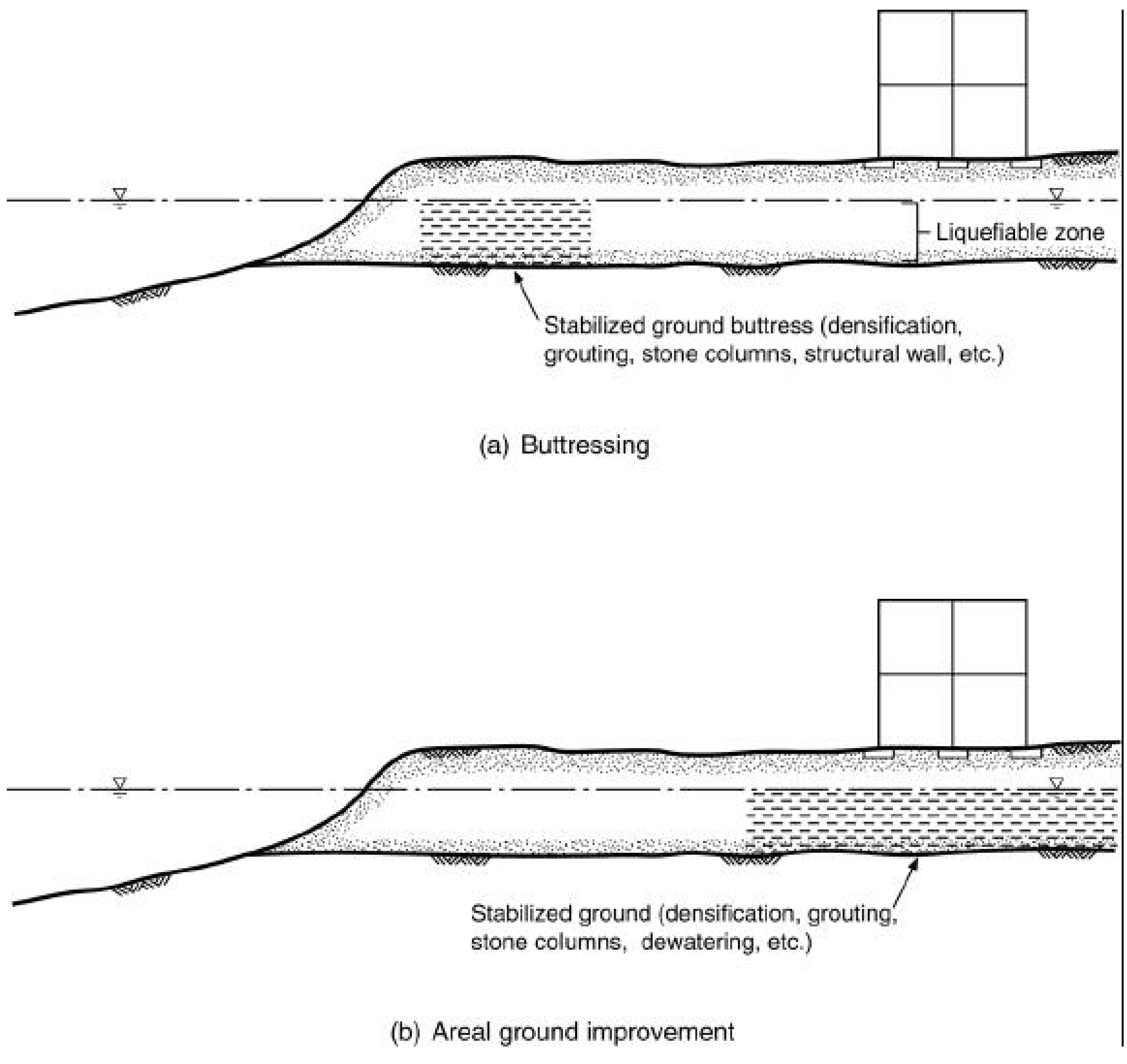


Figure F-23 Conceptual schemes to resist liquefaction-induced lateral spreading.

e. *Landsliding*. If a significant landslide risk to a facility exists, it is generally difficult to design the structure or its foundation to withstand the landslide movement. Mitigating measures typically involve some form of slope stabilization, such as regrading, buttressing, subsurface drainage, or ground modification. If a hazard exists to a structure from rockfalls or shallow soil flows on a slope above the structure, mitigating measures include removal of the material susceptible to failure, buttressing or other stabilization to prevent failure, or creating walls or earth berms to catch or deflect falling rocks or soil flows.

f. *Flooding*. If the depth and velocity of water associated with flooding is not too great, the hazard can be mitigated by creating walls or breakwaters to prevent the water from reaching the structure or dissipating its energy. For floodwaters substantially above the facility elevation or moving with great velocity, resiting may be the only feasible alternative to mitigate the hazard.

F-6. Documentation of Geologic Hazards Evaluations

The methods employed for evaluating geologic hazards, the results of the evaluations, and the conclusions should be documented in a report prepared by the geotechnical professional.